Channel Rehabilitation: Processes, Design, and Implementation

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Channel Rehabilitation:

Processes, Design, and Implementation

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CHAPTER 1

INTRODUCTION

Channel modification or channelization activities are listed among the top 10 sources for non-point pollution impacts to rivers (U.S. Environmental Protection Agency (EPA), 1993). Activities such as straightening, widening, deepening, and clearing channels of debris generally fall into this category. These activities can severely impact major river projects such as navigation and flood control, as well as alter or reduce the diversity of in-stream and riparian habitats.

River systems maintain stability by providing just the necessary flow to transport the available water and sediment. When this balance of water and sediment transport discharge is upset by channelization projects, the system will adjust by increasing or decreasing erosion from the channel bed or river banks. This is a complex interaction that involves the entire watershed and river system. Therefore, a system-wide approach must be taken to analyze these impacts and develop remedial measures. Because of the complexity of channel response to modifications, channel rehabilitation efforts must include many different disciplines such as biology, engineering, geomorphology, geology, and hydrology.

A simple, rigid approach to addressing channel rehabilitation projects is not available. There are too many variables that must be addressed for a one-size-fits-all approach to channel modification activities. Because different river systems vary in geology, climate, ecology, hydrology, and hydraulics; methods utilized in one location may not be applicable to another location. A generalized systematic approach to addressing channel rehabilitation is needed to address the large variety of projects that may range from localized erosion problems that can be addressed using a simple reference reach methodology, to severe basin-wide problems that require a concentrated analysis and design effort. At this time there are no published, definitive guidance or criteria for designing a channel rehabilitation project. This manual provides the methodology and procedures for initiating, planning, evaluating, analyzing, and ultimately designing a channel rehabilitation project.

1.1 PURPOSE

At first glance, the impact of channel modification activities may seem to be confined to specific reaches of the river. It may seem logical that post-channel modification impacts such as

localized erosion or aggradation of sediment in specific locations can be quickly remedied and the problem will go away. Before this assumption can be made, it is wise to consider the impact on the entire system through comprehensive evaluation and analysis. This manual will provide general guidance for system-wide channel rehabilitation by introducing basic fundamentals of geomorphology and channel processes, along with fundamental engineering design methods for performing background evaluations, conducting field data investigations, evaluating channel stability, and producing stable channel designs.

1.2 SCOPE

Considerable information has been published on methods to stabilize channels and banks. The intent of this manual is to provide a systematic basin-wide approach to channel rehabilitation, with emphasis on the basic information and procedures needed to perform an analysis and preliminary design of the channel rehabilitation project. Specific alternative designs for structures, and bank stabilization techniques are not presented. References are provided in the text to give the reader a source of information for detailed design. The project design process presented herein represents an attempt to provide the user with guidance for addressing the impacts of channel modification projects.

CHAPTER 2

THE PROCESS

The diversity of channel rehabilitation projects precludes the acceptable use of a rigid blueprint approach to rehabilitation design. Methods utilized in one ecological, hydrological, or geological setting may not apply to another location. Different goals may require entirely different designs for the same setting. The intent of this chapter is to provide four flowcharts that have been found useful in thinking through the process of channel rehabilitation design.

The distinction between rehabilitation and restoration may be insignificant, depending on existing conditions. Rehabilitation may be defined as maximizing the potential beneficial uses of resources, to some reasonable and practical level. Restoration is defined as bringing a resource back to some former condition. For this manual, rehabilitation is used in a broad sense that encompasses all aspects of channel modification to achieve some desired improvement goal, whether for complete channel restoration, flood control, navigation, water supply, channel stability, sediment control, or some other beneficial use. Regardless of the goals of the rehabilitation project, the basic fundamentals of planning activities must be followed. A typical planning process was outlined by Jensen and Platts (1990) in the following general steps:

1. Preliminary planning to establish the scope, goals, preliminary objectives, and general approach for restoration;

2. Baseline assessments and inventories of project location to assess the feasibility of preliminary objectives, to refine the approach to restoration, and to provide for the project design;

3. Design of restoration projects to reflect objectives and limitations inherent to the project location;

4. Evaluation of construction to identify, correct, or accommodate for inconsistencies with project design; and

5. Monitoring of parameters important for assessing goals and objectives of restoration.

Based on these guidelines, a systematic approach to initiating, planning, analyzing, implementing, and monitoring of channel rehabilitation projects is presented in the following sections. Although implementation, which includes detail engineering design, construction, and inspection, is briefly presented in these guidelines; detail design is beyond the scope of this manual. Construction and inspection guidelines are usually specific to the agency funding the construction and are not embodied in this manual.

Four flowcharts are presented to introduce the generalized methodology of channel rehabilitation, analysis, the systems approach, and for preliminary design. The intent of the flowcharts is to lead the reader through the process of project initiation, analysis, and design, and in the following Chapters 3 through 6 the intent is to provide the reader with a discussion of the methods to be used.

2.1 THE GENERALIZED PROCESS

The generalized flowchart for channel rehabilitation design is presented in Figure 2.1. The flowchart describes the complete flowchart from initiation to monitoring of the final constructed project. Steps in the generalized flowchart are discussed in the following sections, and, in some cases, separate steps are expanded to autonomous flowcharts.

2.1.1 INITIATION

The initiation of the project must include organization of a team, problem identification, and establishment of goals. The team members must comprise a group who is knowledgeable of the potential beneficial uses of the site, and of the techniques and costs associated with rehabilitation. For example, biologists, geomorphologists, engineers, recreation specialists, and representatives of the adjacent landowners and the community are typical of team members required for a channel rehabilitation planning team.

The number of team members should be as small as reasonably possible, which can be expanded as a viable project is identified and consideration of additional factors is necessary. For example the initial team may be very technically oriented in geology, biology, and engineering. As economic and social factors become relevant, additional members of the community and local government are essential.

Problem identification is an essential step in the initiation phase. Identification of the interrelationship of the problems is an essential feature of a system analysis methodology. By understanding the complete watershed, reliable solutions can be identified. Typically, watersheds have the following types of problems:

a. Upland watershed erosion;
b. Channel incision and bank instability;
c. Agricultural and urban flooding;

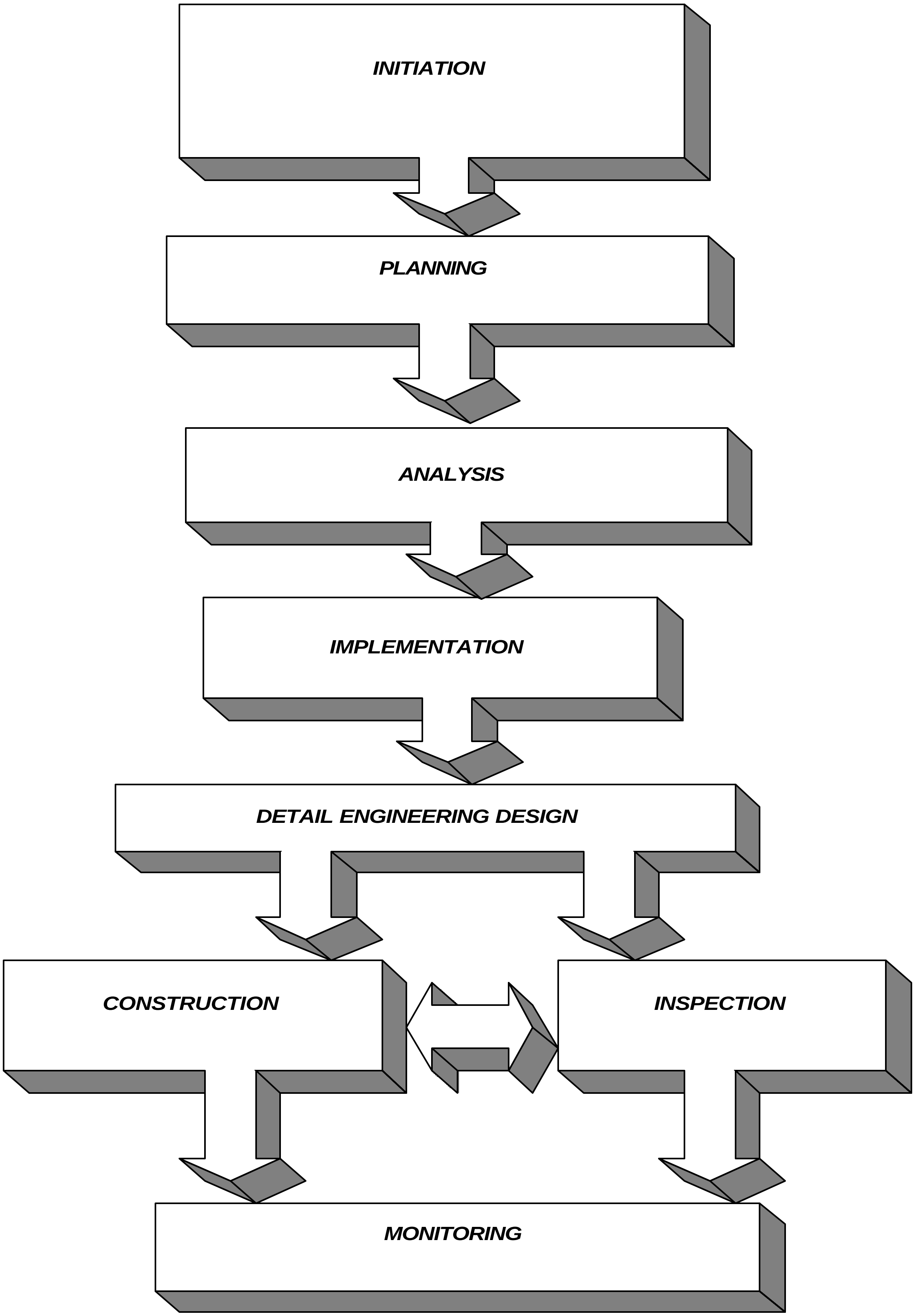


Figure 2.1 Generalized Process for Channel Rehabilitation Design

d. Sedimentation of agricultural land, wetlands, and reservoirs;
e. Damage to stream-related infrastructure; and
f. Destruction of riparian habitat.

Probably the most certain path to an unsuccessful project is to fail to set a clear understanding of the goals of the project. Goals provide the measure of success, and without unambiguous, measurable goals the project cannot succeed.

2.1.2 PLANNING

Planning of the rehabilitation process, the second major element of Figure 2.1, requires the definition of the project in terms of the size of the project, the time required for planning, design, and construction, and in terms of fiscal limits of the proposed project. If the fiscal limits are too confining, no project may be possible, or the goals of the project may be minimized to achieve only a limited goal. Community input and review should seek to ascertain additional goals, concerns, and resources. The planning process is the subject of many texts and papers, and it is not the intent of this manual to cover this subject in detail. Suggested readings are *Restoration of Aquatic Ecosystems* (National Academy of Sciences, 1992), *Water Resources Planning* (Grigg, 1985), and *How to Save a River, A Handbook for Citizen Action* (Bolling, 1994). The primary purpose of the planning process for the subjects addressed in this manual is to have definite, identified goals before the project design begins.

2.1.3 ANALYSIS

Analysis is the third major element of Figure 2.1. Analysis to support channel rehabilitation projects involves: 1) evaluation of alternatives to reach project goals, 2) a systematic approach to assimilation of the data and information necessary to make informed design decisions, and 3) the preliminary design process. Figure 2.2 presents the analysis approach as a sub-element of Figure 2.1, and Figure 2.2 will be discussed in Section 2.2.

2.1.4 IMPLEMENTATION

The next step in the flowchart is the implementation of the project (Figure 2.1). The major elements included in implementation are detail engineering design, construction, and inspection. Detail engineering design would include, for example, computations of riprap size, structural design of drop structures, design of safety features, or other specific details requiring engineering design and construction drawings. Construction and inspection are not included in this manual, and are generally included in standard guidelines provided by the agency funding the project.

2.1.5 MONITORING

Monitoring of stream rehabilitation measures is essential for establishing requirements for maintenance and repair of features, for establishing performance of the measures, and for providing an essential feedback loop to planning and design of future projects. For example, if habitat enhancement is a goal of the project, sampling of the biota is the only true measure of success. In addition, constructed features should be monitored to determine if the features are performing as expected. Major watershed and stream rehabilitation projects may require several years to construct, and monitoring of the earlier constructed portions of the project can be directly related to improvements in the later portions of the project.

2.2 THE ANALYSIS PROCESS

The analysis process, as shown in Figure 2.2, requires project goals to be defined prior to the process. Three key elements in the analysis process are:

a. to evaluate the potential alternatives that may satisfy project goals;
b. to implement a systems approach that encourages the development of a thorough understanding of the watershed physical processes; and
c. to develop a preliminary design that satisfies project goals.

2.2.1 GOALS

The goals developed for the project in planning dictate the progress of the rehabilitation design and the alternatives which need to be evaluated to achieve the selected goals. Goals that may be selected include reduction in downstream sediment delivery, channel stabilization to reduce erosion and to provide better riparian habitat, flood control, or others.

2.2.2 EVALUATION OF POTENTIAL ALTERNATIVES

To satisfy project goals, alternatives should be considered early in the process. These alternatives may include structural designs to mitigate erosion or enhance stability, changes in operational methods along the stream, or land use. For example, alternatives to provide flood control usually can be described as convey, control, or confine, i.e., include improved flood conveyance by increase channel size or slope, control of flood discharge by reservoirs upstream or land use change, or confining the flood by using levees. Chapter 3 discusses the geomorphic and engineering principles that should be considered in evaluating alternatives.

Chapter 4 of this manual presents a summary of channel modification activities which generally include the effects of the above described conditions. These activities fall under the general project categories of flood control, drainage, navigation, sediment control, infrastructure protection, channel bank stability, and flow control. Channel modification activities such as channel

cleanout, snagging and clearing, channel enlargement, channel realignment, dredging, diversions, and dams are presented. For each activity, the hydraulic effects and environmental impacts are discussed, along with suggested remedial practices and project operations and maintenance procedures.

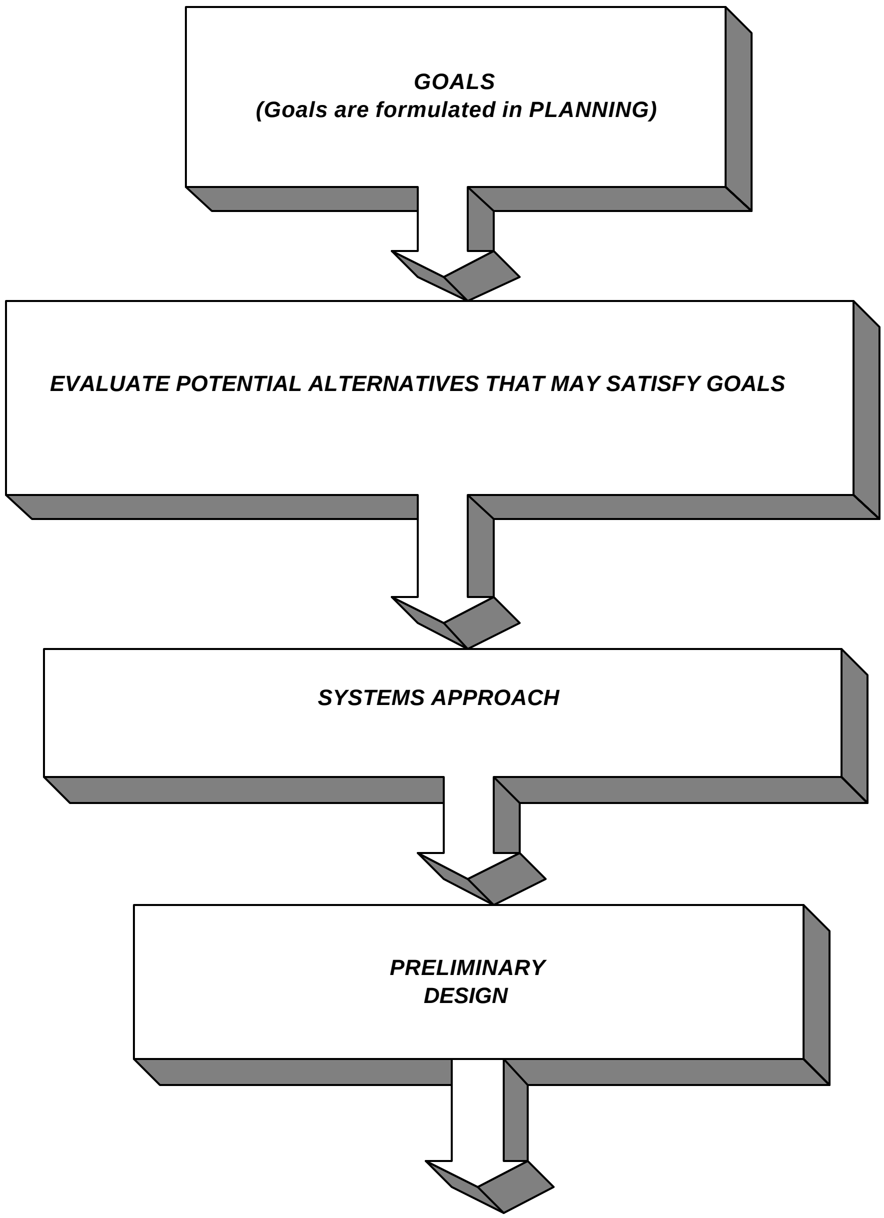


Figure 2.2 The Analysis Process for Channel Rehabilitation Design

2.2.3 SYSTEMS APPROACH

The systems approach, a sub-element of Figure 2.2, provides an approach to evaluating channel and watershed processes in preparation for preliminary design of the project. A systems approach must be used to identify and solve interrelated problems of a watershed. For this approach, no single reach of the system can be viewed as an isolated system. Solutions to problems must be approached in an integrated fashion. The systems approach provides a process-based framework to define watershed dynamics and to develop composite solutions. Understanding the physical process occurring in the watershed is the only rational basis for developing a long-term solution.

The systems approach describes the qualitative and quantitative methods for background and field investigations that are critical to approaching channel rehabilitation design. This approach was developed because watersheds have a number of interrelated problems that require an integrated solution. Development of this approach provides a process-based framework to define watershed and channel dynamics and to develop integrated solutions. An understanding of the system dynamics is essential to assess the consequences of a proposed rehabilitation project.

2.2.4 PRELIMINARY DESIGN

At this point in the Analysis flowchart (Figure 2.2), goals have been established, potential alternatives have been considered and a thorough understanding of the physical processes in the watershed has been developed. Next in the channel rehabilitation design sequence is the preliminary design. The primary goal of the preliminary design section is the computation of a stable channel morphology and development of a preliminary design to meet project goals. The preliminary design phase takes the data and information generated from the initiation, planning, background investigations, field investigations, and computational methodologies, and initiates the design process. The preliminary design process will ultimately resolve the question: Are project goals met?

2.2.5 SUMMARY OF THE ANALYSIS CHART

The three steps of the analysis process are:

1. evaluate alternatives to achieve the project goals;
2. develop a thorough understanding of the watershed in the systems analysis; and
3. develop the preliminary design of the project.

The results of the Analysis process progresses to Implementation, which includes engineering design, plans and specifications, construction and inspection, and monitoring.

2.3 SYSTEMS APPROACH

The systems approach flowchart, a primary component of the analysis process, is presented in Figure 2.3 and is explained in Chapter 5 of this manual. The central components of the systems approach include background investigations, field investigations, and stable channel design methodologies.

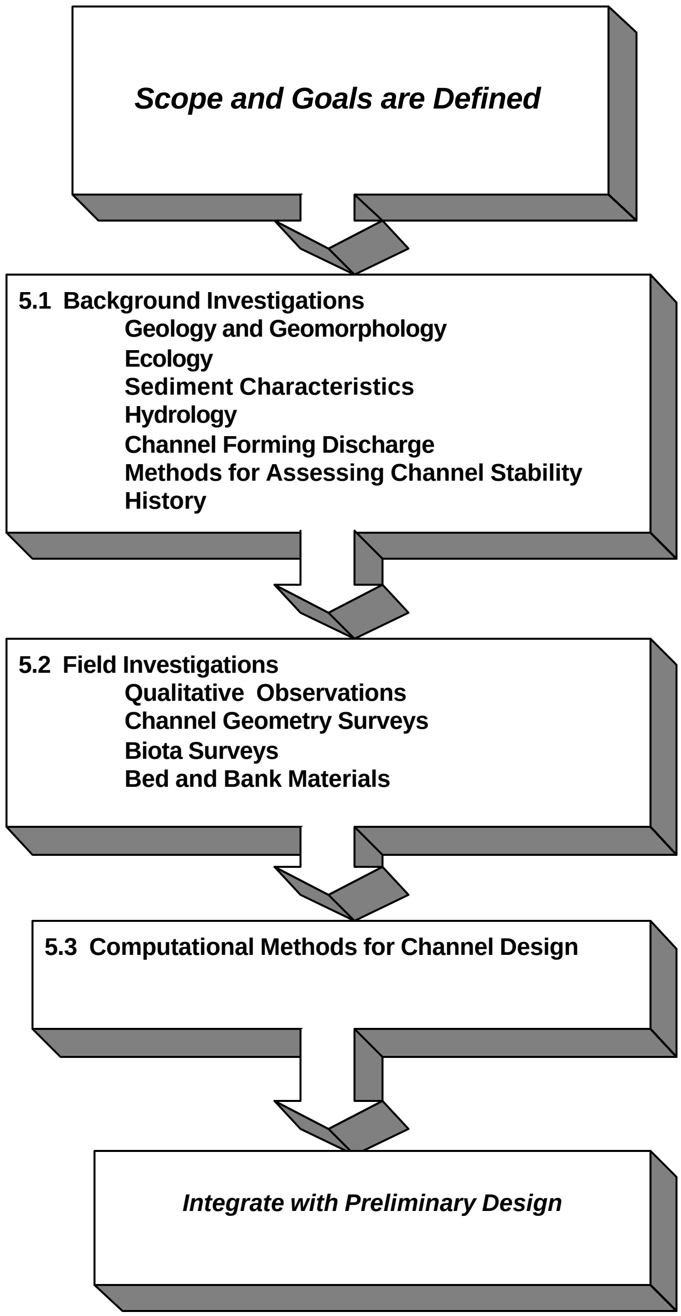


Figure 2.3 The Systems Approach

Background investigations include assimilating data and information in areas such as watershed geology, sediment, geography, hydrology, and historical channel stability. Background investigations provide a historical perspective in which past behavior is examined to provide some indication of the future trends in channel adjustments.

Geomorphic assessments provide the process-based framework to define past and present watershed dynamics, develop integrated solutions, and assess the consequences of remedial actions. This is an essential part of the design process, for either local stabilization treatment or development of a comprehensive plan for an entire watershed.

The existing-condition status of the watershed and channel is determined through field investigations. The field tasks focus on the study area. These tasks involve assimilating the necessary field samples, data, and observations that are required to support analysis techniques. These investigations provide a qualitative description of existing channel processes and bank characteristics, as well as quantitative data related to sediment characteristics (types, gradations, and transport), channel hydraulics, and system stability.

Ultimately, the background and field investigation data are combined with computational methods for stable channel design. The result of the initial cycle through the systems approach provides information on channel hydraulics and sediment transport characteristics, watershed dynamics, and system stability necessary to begin the preliminary design phase of channel rehabilitation.

2.4 PRELIMINARY DESIGN

Figure 2.4 describes the sequence of events for preliminary design. At this juncture, the initial stable channel design from the systems approach is evaluated against proposed goals. If goals are satisfied by the existing condition design, then no further work is required, and the work can proceed to the design of local stabilization and habitat enhancement features. If the goals are not satisfied due to system instability or a need to modify design parameters to meet project goals, an iterative design process is initiated in which design parameters such as channel forming discharge and stable channel dimensions are re-evaluated, and measures for restoring stability such as grade control, bank stabilization, and planform properties are considered.

The preliminary design phase uses the data, information, findings, and analysis techniques presented in Chapter 5 to formulate a stable channel design that will meet project goals. As shown in the preliminary design process flow chart (Figure 2.4), the process can be limited in scope if it is determined, after background and field investigations, that the system is stable and project goals can be met without additional design efforts. At this point, detailed design procedures can commence. More often than not, this is not the case. If, upon initial investigations, channel instabilities are present, or project goals cannot be accomplished with the existing conditions, a preliminary design process must be undertaken to develop a channel design which will achieve the desired goals.

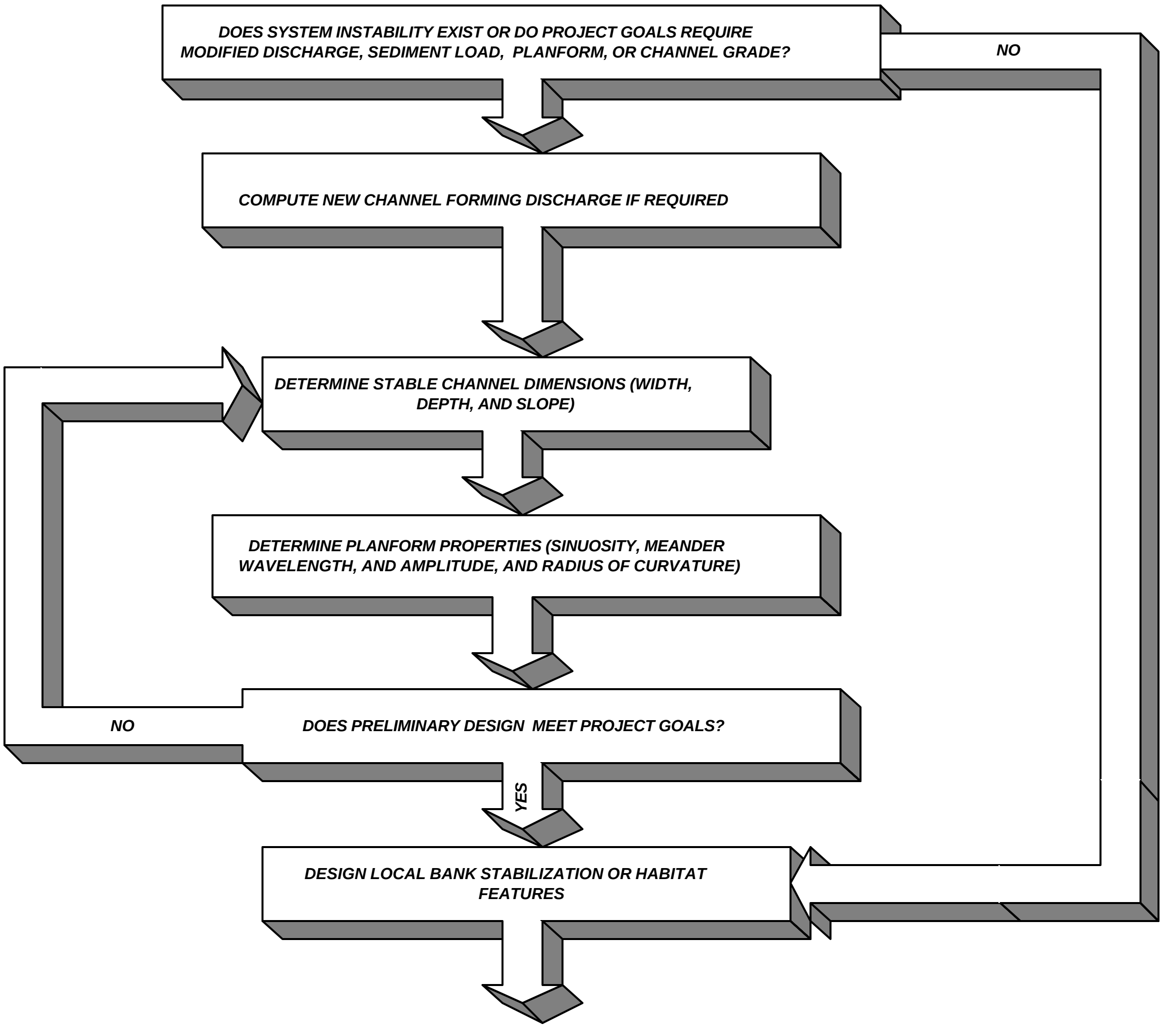


Figure 2.4 Preliminary Design for Channel Rehabilitation

After the geomorphic assessment is completed, the preliminary design process (Figure 2.4) is initiated. A critical question is asked based on results from the geomorphic assessment. Does system instability exist or do project goals require modified discharge, modified sediment yield, modified planform, or modified channel grade? If the answer to all elements of the question is no, then the project may require only localized treatments such as local bank stabilization or local habitat enhancement features. If the answer to any of the elements of the question is yes, then a four step procedure with a feed back loop is required (Figure 2.4). The first of these three steps is to compute a new channel forming discharge, if needed. The second step is to determine stable channel dimensions of width, depth, and slope that meet project goals of water and sediment conveyance. Empirical and computational methods for determining stable channel morphology are discussed in Sections 5.3.2 - 5.3.8 of Chapter 5. The third of the four steps is to implement the proper slope by determining planform properties of sinuosity, meander wavelength, amplitude, and radius of curvature. The fourth step in this feed back loop simply asks the question "does

the plan meet project goals"? If the answer is yes, the process moves to the consideration of local stabilization or local habitat features. If the goals are not met, the process then proceeds back to the first step in the loop and repeats the design process.

It must be recognized that there is no single approach that is universally applicable to all projects. The specific methodology must reflect the project goals, financial and man-power resources, and environmental considerations. For this reason, the design approach discussed herein should not be viewed as a "cookbook" process, but rather, as a logical strategy that can be adapted to the specific demands of the project.

In this section, methodologies will be introduced and discussed which follow this generalized approach. A step-by-step procedure is presented for each methodology which will instruct the user on how to approach preliminary design for channel rehabilitation.

2.4.1 PRELIMINARY DESIGN METHODOLOGY - DETERMINATION OF SYSTEM STABILITY

Whenever, a channel rehabilitation project is contemplated, one of the first issues that must be addressed is the overall stability of the channel system being considered. Failure to consider the system stability frequently results in costly channel designs that fail to function properly, both from an engineering and environmental perspective. The determination of the stability of the channel system is accomplished through the geomorphic approach discussed in Chapter 5. If the project goals are simply the stabilization of localized instabilities, or the installation of localized habitat features, then it is important that system-wide channel instability does not exist. If it is determined that system-wide instabilities do not exist, then the design of the local bank stabilization or habitat features can proceed. However, if system-wide instability exist, in the form of aggradation, degradation, or plan form changes, then it becomes necessary to first address these system instabilities before local stabilization is considered. Likewise, if the system is currently in dynamic equilibrium, but, if it is anticipated that the project will modify this stability, then a more rigorous systems analysis must be performed.

2.4.2 PRELIMINARY DESIGN METHODOLOGY - COMPUTATION OF CHANNEL FORMING DISCHARGE

The second step in the preliminary design sequence is the determination of the channel forming discharge. The procedures for determining this value were discussed in Chapter 5. If the project goals will not significantly modify the water and sediment yield, then the existing channel forming discharge calculated during the geomorphic assessment can be used for the design process. For example, if the objectives of the rehabilitation project are to simply stabilize the channel grade using low drop grade control structures, then it may not be necessary to calculate a new post-project channel forming discharge, since the effect of these structures would not be expected to dramatically change the water and sediment loads. However, if the plan involves more comprehensive features such as flow control, channel enlargement, flow diversions,

or other improvements that would significantly alter the post-project water and sediment loads, then it may be necessary to calculate a new channel forming discharge. In these instances, the new channel forming discharge would have to be calculated using the effective discharge analysis, or a specified recurrence interval flow based on post-project conditions. The use of the bankfull discharge would not be appropriate in these situations, because the bankfull morphology would reflect pre-project morphology, and would not be correlated with the post-project flows. The new post-project channel forming discharge is used in the determination of the stable channel dimensions discussed in the next step.

2.4.3 PRELIMINARY DESIGN METHODOLOGY - DETERMINATION OF STABLE CHANNEL DIMENSIONS

Computation of stable channel dimensions can be accomplished with a number of channel design methods (Chapter 5, Sections 5.3.2 - 5.3.7). The selection of the appropriate method is a function of a number of factors such as level of study (reconnaissance, feasibility, detailed design, etc), funding and time constraints, complexity of project and stream characteristics, consequences of failure of the design, and available data. For instance, during early recon studies, it may be appropriate to utilize some of the less computationally intense empirical methods. However, as the level of study increases, it might be necessary to conduct more rigorous analyses using SAM, HEC-6, or other numerical methods. It should also be noted that in some situations it may not be necessary to calculate all three variables. For example, if the rehabilitation plan simply calls for the layout of a series of grade control structures to stabilize the channel, then it may only be necessary to calculate a stable slope to be used in spacing the structures.

The following is a brief summary of the applicability of the various methods discussed in Sections 5.3.2 through 5.3.7.

Maximum permissible velocity and tractive force. The maximum permissible velocity and tractive force design methods were discussed in Chapter 5, Sections 5.3.2 and 5.3.3. These methods are most applicable to reconnaissance level studies where the purpose is to quickly assess various alternatives. Neither of these methods specify a complete design channel design because they can be satisfied by various combinations of width, depth and slope. Additionally, these two methods are generally not applicable to situations where a significant bed material load exists.

Regime and hydraulic geometry. The regime theory of channel design is based on empirical relationships developed from field data collected from numerous river and canal systems (Chapter 5, Section 5.3.4). The USACE regime method (USACE, 1994) recommends using locally or regionally developed equations for channel design. If this is not available, graphs are provided for estimating width, depth, and slope of the channel given the channel forming discharge and bed material description. As with the permissible velocity and tractive force methods, the regime approach is more suited to reconnaissance level studies, and caution is advised when attempting to use these methods for detailed design. The reader is referred to the limitations discussed in Section 5.3.4.

In all of the above described channel design methods, channel dimensions are either assumed or iteratively varied to meet design criteria such as allowable velocities, tractive force, or regime relations.

Analytical design methods. As noted above, the empirical design methods may be appropriate for reconnaissance level studies, or small rehabilitation projects. However, if the projects are large scale, or involve significant bed material transport then these methods are not generally suitable, and it may be necessary to adopt one of the more rigorous analytical design procedures discussed in Chapter 5, Section 5.3.5. While these procedures address more fully the dominant processes in the channel system, it must be recognized that the data requirements are also much more intensive.

2.4.4 DETERMINE A STABLE CHANNEL MEANDER WAVELENGTH FOR THE PLANFORM

In some instances, the project goals may require modification of the existing planform. When this occurs, the meander planform properties must be designed to be compatible with the stable channel dimensions calculated in the previous step (Section 2.4.3).

The most reliable hydraulic geometry relationship for meander wavelength is wavelength vs. width. As with the determination of channel width, preference is given to wavelength predictors from stable reaches of the existing stream either in the project reach or in reference reaches. Lacking data from the existing stream, general guidance is available from several literature sources (e.g. Leopold *et al.*, 1964). The meander length is computed from the following equation:

$$\text{meander length} = \frac{\text{wavelength x valley slope}}{\text{slope}}$$

2.4.5 PLANFORM LAYOUT USING THE MEANDER WAVELENGTH AS A GUIDE

One way to accomplish this task is to cut a string to the appropriate length and lay it out on a map. Another, more analytical approach, is to assume a sine-generated curve for the planform shape as suggested by Langbein and Leopold (1966) and calculate x-y coordinates for the planform. The sine-generated curve produces a very uniform meander pattern. A combination of the string layout method and the analytical approach would produce a more natural looking planform.

Check the design radius of curvature to width ratio, making sure it is within the normal range of 1.5 to 4.5. If the meander length is too great, or if the required meander belt width is unavailable, grade control may be required to reduce the channel slope. In streams that are essentially straight (sinuosity less than 1.2) riffle and pool spacing may be set as a function of channel width. The empirical guide of 6-10 channel widths applies here, with the lower end for steeper channels and the higher end for flatter channels. Two times this riffle spacing gives the total channel length through one meander pattern.

At this point, a word of caution is needed about re-establishing meanders in a previously straightened reach. While this generally a commendable goal, and one that may be achievable in certain circumstances, it is usually not as simple as is often purported, particularly in large scale projects, or where severe system instability exists or has existed in the past. For this reason, it is important to consider the stability of the reaches immediately upstream and downstream of the project reach. This is an essential, yet often overlooked component of the design process. If the unstable project reach is bounded on the upstream and downstream ends by stable reaches, or if there exist some sort of man-made or geologic controls on both ends of the reach, then the reach may be much more manageable. Consider the example of a channel reach that is undergoing significant channel widening and has been converted from a meandering to a braided channel due to overgrazing, along the reach. If the channel upstream and downstream of this reach is stable, then elimination of the overgrazing problem, and the re-establishment of the old meander pattern may an achievable goal. Now consider a project where the goal is to restore a five mile segment of a 20 mile straightened reach that has experienced 15 feet of degradation. Several approaches could be identified to meet project goals, but each would have inherent problems. For example, simply constructing a new sinuous channel at the existing channel elevation would not re-establish the natural hydraulic connection between the channel and the floodplain. This problem could be overcome by abandoning the old channel and constructing a new channel in the floodplain. However, transitioning into the downstream reach would pose a serious problem that probably would require expensive and possibly environmentally unacceptable grade control structures to drop the flow from the new channel into the old channel.

2.4.6 SEDIMENT IMPACT ASSESSMENT

The purpose of the sediment impact assessment is to assess the long-term stability of the restored reach in terms of aggradation and/or degradation. This can be accomplished using a sediment budget approach for relatively simple projects or by using a numerical model which incorporates solution of the sediment continuity equation for more complex projects.

2.4.7 PRELIMINARY DESIGN METHODOLOGIES - MEETING PROJECT GOALS AND FINAL DESIGN

At this point in the preliminary design, the overall design process is reviewed. The preliminary channel design is evaluated to insure that project goals such as reducing sediment loads, flood control, or environmental enhancements are met. If not, the design process is re-visited to insure the project goals are met with a stable channel design (Figure 2.4). With a satisfactory design, final local and system wide designs can be initiated. Alternatives for satisfying project goals which were identified at project planning stage can be designed and implemented.

2.4.8 PRELIMINARY DESIGN SUMMARY

The preliminary design phase of channel rehabilitation projects represents the culmination of a systematic approach to planning, initiating, and designing channel rehabilitation projects. It is at this point where all the planning, data assembly, and analysis come together for a channel design that will meet project goals, insure system-wide stability, and insure that designed and constructed alternatives will operate as intended without excessive maintenance or replacement before the projected project lifetime.

2.5 CHAPTER SUMMARY

The methodologies outlined in this chapter represent a systematic and organized process for approaching and designing channel rehabilitation projects. These methods have one thing in common: a systematic and comprehensive approach must be taken to solve stream and watershed problems. Although it may initially appear that mitigating localized problems within a channel may be the best solution, this limited approach may have a far reaching impact upstream and downstream of the affected area. An organized, multi-discipline approach is needed to plan and implement background and field study of the watershed and channel system, to evaluate channel and bank stability, to recognize historical and current hydraulic and sediment transport trends, and to develop alternatives to meet project goals. Without a thorough understanding of watershed dynamics and channel processes, designs that are constructed in the rehabilitation process may create unforseen problems.

CHAPTER 3

FUNDAMENTALS OF FLUVIAL GEOMORPHOLOGY AND CHANNEL PROCESSES

3.1 FLUVIAL GEOMORPHOLOGY

Webster's New World Dictionary defines **fluvial** as: *of, found in, or produced by a river or rivers*. The same reference defines **morphology** as: *any scientific study of form and structure, as in physical geography, etc.* With a little guess work, we can correctly extrapolate that fluvial geomorphology is the study of the form and structure of the surface of the earth (geo) as affected by flowing water. Another definition, although given in jest, may be the one most remembered after this next section. *Geomorphology is the triumph of terminology over common sense*. An equally important term is the **fluvial system**. A system is an arrangement of things to form a whole. The primary goal on which we want to focus in this section is that you are working with a system and the complete system must be considered.

3.1.1 BASIC CONCEPTS

Six basic concepts that should be considered in working with watersheds and rivers are: 1) the river is only part of a system, 2) the system is dynamic, 3) the system behaves with complexity, 4) geomorphic thresholds exist, and when exceeded, can result in abrupt changes, 5) geomorphic analyses provide a historical prospective and we must be aware of the time scale, and 6) the scale of the stream must be considered. Is the stream a small, mountain meadow trout stream, or is it the Mississippi River?

3.1.1.1 The Fluvial System

Schumm (1977) provides an idealized sketch of a fluvial system (Figure 3.1). The parts are referred to as:

Zone 1 - the upper portion of the system that is the watershed or drainage basin; this portion of the system functions as the sediment supply.

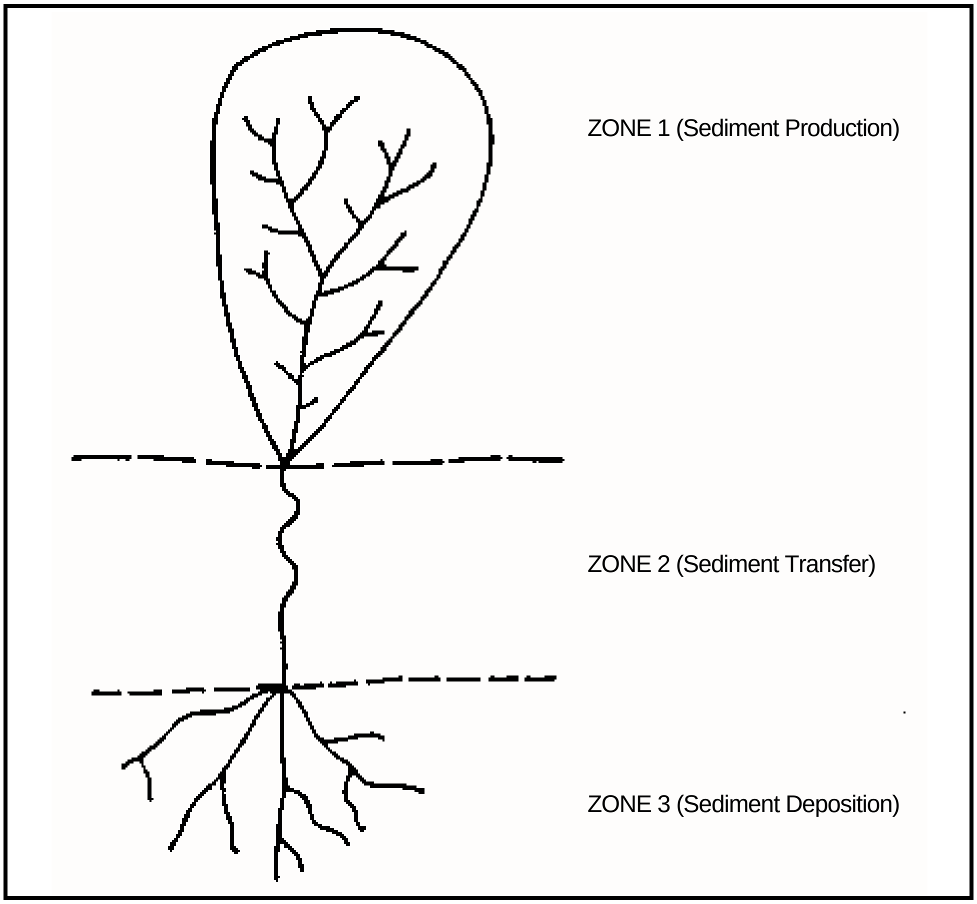


Figure 3.1 The Fluvial System (after Schumm, 1977)

Zone 2 - the middle portion of the system that is the river; this portion of the system functions as the sediment transfer zone.

Zone 3 - the lower portion of the system may be a delta, wetland, lake, or reservoir; this portion of the system functions as the area of deposition.

These three zones are idealized, because in actual conditions sediments can be stored, eroded, and transported in all zones. However, within each zone one process is usually dominant. For our purposes in planning channel stabilization, we are primarily concerned with Zone 2, the transfer zone. We may need to treat only a small length of a stream bank (Zone 2) to solve a local instability problem; however, from a system viewpoint we must insure that our plan does not interfere with the transfer of sediment from upstream (Zone 1) to downstream (Zone 3). In channel stabilization planning we must not neglect the potential effects that may occur throughout the system.

The fundamental concept that a stream is a portion of a large and complex system may have been most eloquently stated by Dr. Hans Albert Einstein:

If we change a river we usually do some good somewhere and "good" in quotation marks. That means we achieve some kind of a result that we are aiming at but sometimes forget that the same change which we are introducing may have widespread influences somewhere else. I think if, out of today's emphasis of the environment, anything results for us it is that it emphasizes the fact that we must look at a river or a drainage basin or whatever we are talking about as a big unit with many facets. We should not concentrate only on a little piece of that river unless we have some good reason to decide that we can do that.

3.1.1.2 The System is Dynamic

In each of the idealized zones described above, a primary function is listed. Zone 1 is the sediment source that implies that erosion of sediment occurs. Zone 2 is the transfer zone that implies that as rainfall increases soil erosion from the watershed, some change must result in the stream to enable transfer of the increased sediment supply. Zone 3 is the zone of deposition and change must occur as sediment builds in this zone, perhaps the emergence of wetland habitat in a lake, then a change to a floodplain as a drier habitat evolves. The function of each zone implies that change is occurring in the system, and that the system is dynamic.

From an engineering viewpoint some of these changes may be very significant. For example, loss of 100 feet of stream bank may endanger a home or take valuable agricultural land. From a geomorphic viewpoint, these changes are expected in a dynamic system and change does not necessarily represent a departure from a natural equilibrium system. In planning stabilization measures, we must realize that we are forced to work in a dynamic system and we must be try to avoid disrupting the system while we are accomplishing our task.

3.1.1.3 Complexity

Landscape changes are usually complex (Schumm and Parker, 1973). We are working in a system and we have defined a system as an arrangement of things to form a whole. Change to one portion of the system may result in complex changes throughout the system.

When the fluvial system is subjected to an external influence such as channelization of part of a stream, we can expect change to occur throughout the system. Channelization usually increases stream velocity and this would allow the stream to transfer more sediment, resulting in erosion upstream and deposition downstream of the portion of the stream channelized. For example, some Yazoo Basin streams in north Mississippi that were channelized in the 1960s responded initially, but an equilibrium has not yet been reestablished as repeated waves of degradation, erosion, and aggradation have occurred.

3.1.1.4 Thresholds

Geomorphic thresholds may be thought of as the straw that broke the camel's back. In the fluvial system this means that progressive change in one variable may eventually result in an abrupt change in the system. If a river erodes a few grains of soil from the toe of the river bank, no particular response will be noticed. If that continues with no deposition to balance the loss, the bank may eventually fail abruptly and dramatically due to undermining. The amount of flow impinging along a bank may vary considerably with no apparent effect on the stabilization; however, at some critical point the bank material will begin to move and disastrous consequences can result.

In this example the change was a gradual erosion of a few grains of soil and a variability of stream velocity, both which could be considered to be within the natural system. This type of threshold would be called an intrinsic threshold. Perhaps the threshold was exceeded due to an earthquake or caused by an ill-planned bank stabilization project. These would be called extrinsic thresholds. The planner must be aware of geomorphic thresholds, and the effect that their project may have in causing the system to exceed the threshold.

Channel systems have a measure of elasticity that enables change to be absorbed by a shift in equilibrium. The amount of change a system can absorb before that natural equilibrium is disturbed depends on the sensitivity of the system, and if the system is near a threshold condition, a minor change may result in a dramatic response.

3.1.1.5 Time

We all have been exposed to the geologists view of time. The Paleozoic Era ended only 248 million years ago, the Mesozoic Era ended only 65 million years ago, and so on. Fortunately, we do not have to concern ourselves with that terminology. An aquatic biologist may be concerned with the duration of an insect life stage, only a few hours or days. What we should be aware of is that the geologist temporal perspective is much broader than the temporal perspective of the engineer, and the biologist perspective may be a narrowly focused time scale. Neither profession is good nor bad because of the temporal perspective; just remember the background of people or the literature with which you are working.

Geomorphologists usually refer to three time scales in working with rivers: 1) geologic time, 2) modern time, and 3) present time. Geologic time is usually expressed in thousands or millions of years, and in this time scale only major geologic activity would be significant. Formation of mountain ranges, changes in sea level, and climate change would be significant in this time scale. The modern time scale describes a period of tens of years to several hundred years, and has been called the graded time scale (Schumm and Lichty, 1965). During this period a river may adjust to a balanced condition, adjusting to watershed water and sediment discharge. The present time is considered a shorter period, perhaps one year to ten years. No fixed rules govern these definitions. Design of a major project may require less than ten years, and numerous minor projects are designed and built within the limitations of present time. Project life often

extends into graded time. From a geologists temporal point of view, engineers build major projects in an instant of time, and expect the projects to last for a significant period.

In river related projects time is the enemy, time is our friend, and time is our teacher. We must learn all we can by adopting a historical perspective for each project that we undertake.

3.1.1.6 Scale

The physical size of the stream may impose limits on the type of planned enhancements to the stream. For example, many variations of anchoring trees along the bank have been successfully used along small and moderate size streams to provide cover and to decrease erosion of the bank. Anchoring of trees along the bank is a reasonable method of stabilization. However, for large rivers that may have bank heights of 30 feet and a yearly water surface elevation fluctuation of 20 to 30 feet, the anchored tree may be an unreasonable method for stabilization. Applications designed for a small stream may not be directly transferrable to larger streams. If we are to transfer techniques for enhancement from stream to stream; we must also understand the design principles of those techniques. Principles, such as increasing the cover and decreasing the water velocity at the water-bank interface are transferable; however, the direct technique may not be transferable.

3.1.2 LANDFORMS

Now it is time to give you a brief introduction into what you may see when you go to the field. The following discussion will be confined primarily to depositional landforms along meandering rivers, and a little information concerning terraces.

A **floodplain** is the alluvial surface adjacent to a channel that is frequently inundated (Figure 3.2). Although much of the literature until the 1970s suggested that the mean annual flood was the bankfull discharge, Williams (1978) clearly showed that out of thirty-five floodplains he studied in the U.S., the bankfull discharge varied between the 1.01- and 32-year recurrence interval. Only about a third of those streams had a bankfull discharge between the 1- and 5-year recurrence interval discharge. Knowledge of alluvial landforms will allow a more informed determination of bankfull than depending solely on the magnitude of the flood.

Table 3.1 and Figure 3.2 together provide a quick summary of some alluvial landforms found along a meandering stream. From the perspective of a stream stabilization planner, it is extremely important to know that all the materials along the bank and in the floodplain are not the same. The materials are deposited under different flow conditions.

For example, **backswamps** and **channel fills** will usually be fine-grained and may be very cohesive. This is because both landforms are deposited away from the main flow in the channel, in a lower energy environment. **Natural-levee** deposits are coarser near the channel and become finer away from the channel as the energy to transport the larger particles dissipates.

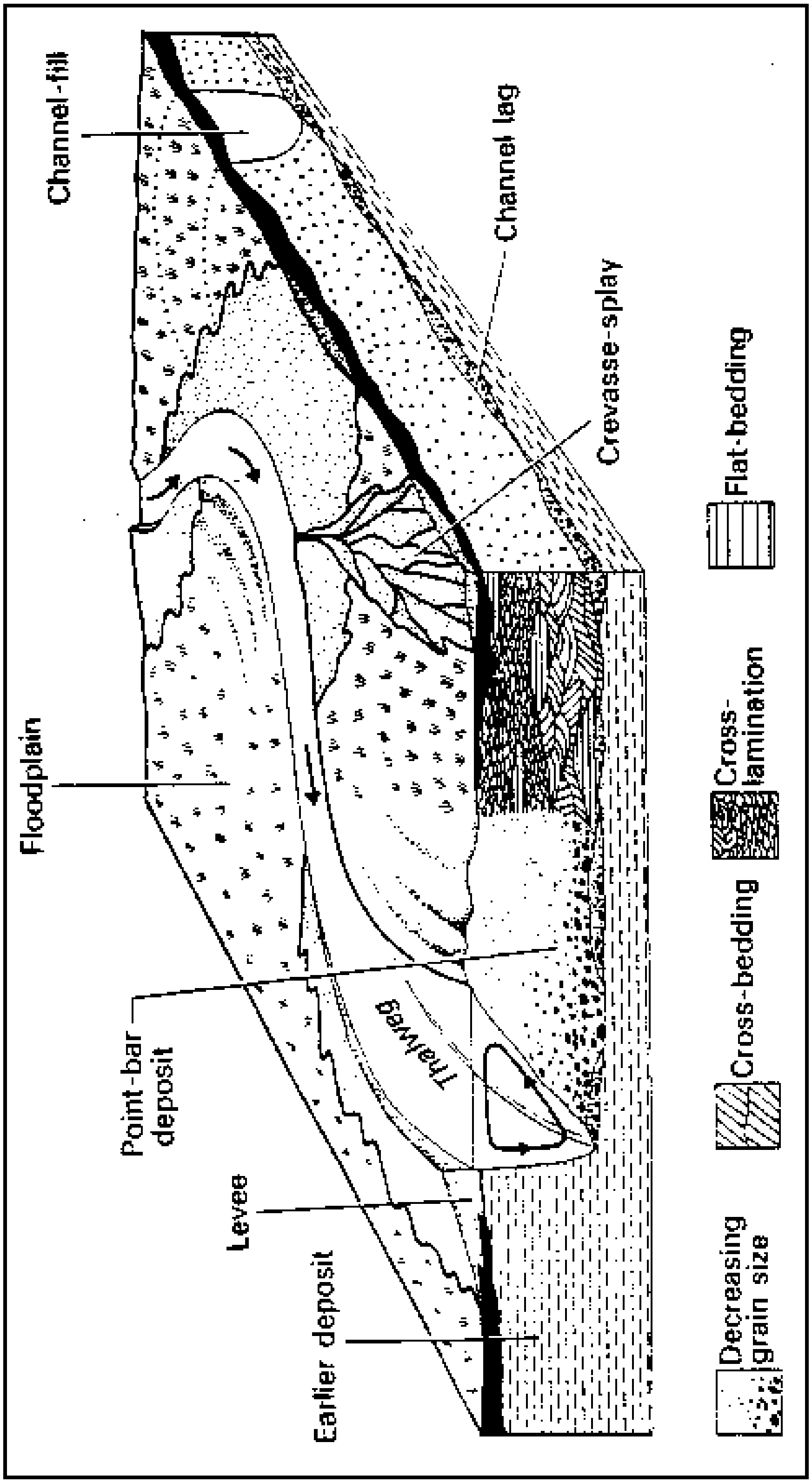


Figure 3.2 Landforms for a Meandering River (Collinson, 1978 after Allen, 1970)

Table 3.1 Classification of Valley Sediments

Place of Deposition	Name	Characteristics
Channel	Transitory channel deposits	Primarily bedload temporarily at rest; for example, alternate bar deposits.
	Lag deposits	Segregation of larger of heavier particles, more persistent than transitory channel deposits, and including heavy mineral placers.
	Channel fills	Accumulations in abandoned or aggrading channel segments, ranging from relatively coarse bedload to plugs of clay and organic muds filling abandoned meanders.
Channel margin	Lateral accretion deposits	Point and marginal bars which may be preserved by channel shifting and added to overbank floodplain by vertical accretion deposits at top; point-bar sands and silts are commonly trough cross-bedded and usually form the thickest members of the active channel sequence.
Overbank flood plain	Vertical accretion deposits	Fine-grained sediment deposited from suspended load of overbank floodwater, including natural levee and backswamp deposits; levee deposits are usually horizontally bedded and rippled fine sand, grading laterally and vertically into point-bar deposits. Backswamp deposits are mainly silts, clays and peats.
	Splays	Local accumulations of bedload materials, spread from channels on to adjacent floodplains; splays are cross-bedded sands spreading across the inner floodplain from crevasse breaches.
Valley margin	Colluvium	Deposits derived chiefly from unconcentrated slope wash and soil creep on adjacent valley sides.
	Mass movement deposits	Earthflow, debris avalanche and landslide deposits commonly intermix with marginal colluvium; mudflows usually follow channels but also spill overbank.

Point bars represent a sequence of deposition in which the coarser materials are at the bottom and the finer materials at the top. From the viewpoint of the channel stabilization planner, the more erosion resistant materials may then be silts and clays deposited at the top and very erosive sand may comprise the toe of the slope. Therefore, if the channel you are attempting to stabilize is eroding into an old point bar deposit, you may encounter several problems. Along the same line of thinking, an abandoned channel fill may appear on the eroding bank as a clay plug.

Different types of bank instability can also arise depending on how the materials were deposited. Consider a point bar deposit with a sandy base that has been deposited over a backswamp clay deposit. This can result in sub-surface flow at the sand-clay interface that can cause the granular material to be washed out of the bank and failure to occur some distance back from the channel. Stabilization could include proper drainage of the top of the bank to deprive the failure mechanism of the percolating groundwater source.

In addition to the landforms briefly described in Table 3.1, we should introduce **terraces**. Terraces are abandoned floodplains formed when the river flowed at a higher level than now (Ritter, 1978). Terraces are produced by incision of the floodplain (Schumm, 1977). In other words, the stream channel has down cut leaving the previous floodplain, and is establishing a new, lower floodplain. The appearance of a terrace or a series of terraces in a surveyed cross-section may be as broad stair steps down to the stream. The steps may be broad and continuous throughout the length of the stream segment, or may be discontinuous and could be only a few feet in width.

3.1.3 RIVER MECHANICS

River mechanics is the subset of both fluvial geomorphology and open channel hydraulics which focuses on the form and structure of rivers. Specifically, it address the channel pattern, channel geometry

(cross section shape), planform geometry, and the channel slope. The purpose of this section is to introduce you to some of the basic characteristics of rivers, and help define some of the confusing terminology you may encounter when dealing with rivers.

Rivers and streams are dynamic and continuously change their position, shape, and other morphological characteristics with variations in discharge and with the passage of time. It is important not only to study the existing river but also the possible variations during the lifetime of the project, particularly in terms of effective treatment of bank erosion. The characteristics of the river are determined by the water discharge, the quantity and character of sediment discharge, the composition of the bed and bank material of the channel, geologic controls, the variations of these parameters in time, and man's activities. To predict the behavior of a river in a natural state or as affected by man's activities, we must understand the characteristics of the river as well as the mechanics of formation.

3.1.3.1 Channel Pattern

Channel pattern describes the **planform** of a channel. The primary types of planform are meandering, braided, and straight. In many cases, a stream will change pattern within its length. The type pattern is dependent on slope, discharge, and sediment load.

The most common channel pattern is the **meandering stream** (Figure 3.3). A meandering channel is one that is formed by a series of alternating changes in direction, or bends. Relatively straight reaches of alluvial rivers rarely occur in nature. However, there are instances where a reach of river will maintain a nearly straight alignment for a long period of time. Even in these relatively straight reaches, the thalweg may still meander and alternate bars may be formed. Straight streams generally occur in relatively low energy environments. The **braided pattern** is characterized by a division of the river bed into multiple channels (Figure 3.4). Most braided streams are relatively high gradient and relatively coarse streams.

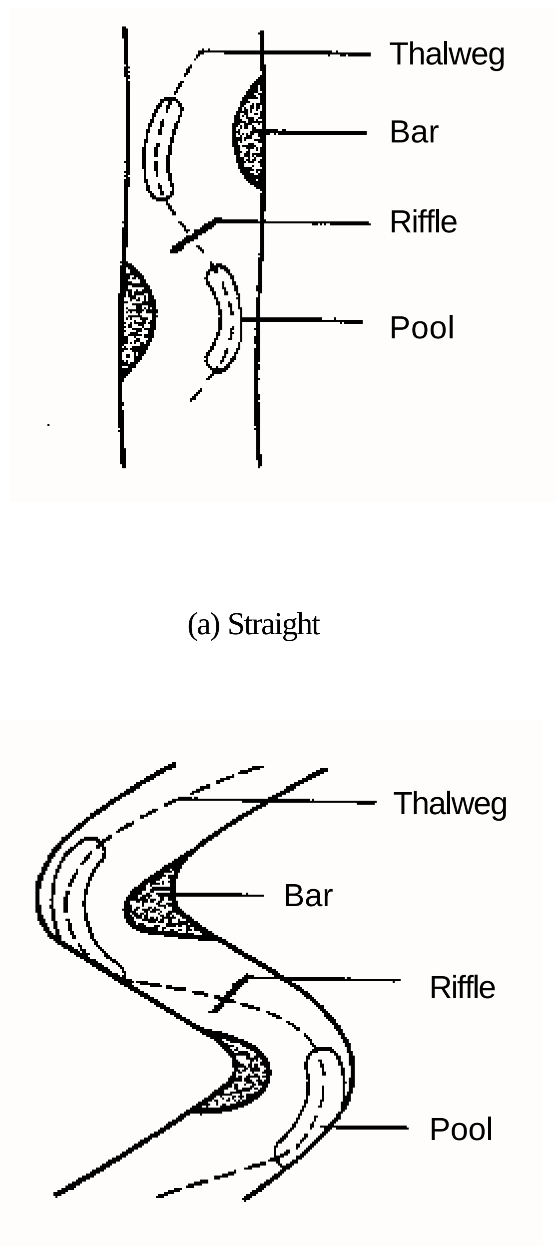
3.1.3.2 Channel Geometry and Cross Section

The following paragraphs describe the channel geometry and cross sectional characteristics of streams. Since meandering streams are the most common form of alluvial channels this discussion will focus primarily on this stream type.

Pools and Crossings. A schematic showing features associated with meanders and straight channels is given in Figure 3.5. As the **thalweg**, or trace formed by the deepest portion of the channel, changes from side to side within the channel, the momentum of the flow affects the cross-sectional geometry of the stream. In bends, there is a concentration of flow due to centrifugal forces. This causes the depth to increase at the outside of the bend, and this area is known as a **pool**. As the thalweg again changes sides below a bend, it crosses the centerline of the channel. This area is known as the **riffle or crossing**. At the point of tangency between adjacent bends, the velocity distribution is fairly consistent across the cross section, which is approximately rectangular in shape.

Figure 3.3 Typical Meandering River

Figure 3.4 Typical Braided River



(b) Meandering

Figure 3.5 Features Associated With (a) Straight and (b) Meandering Rivers

The concentration of flow in the bend is lost and the velocity decreases accordingly, thus causing deposition in the crossing.

Cross Section Shape. The shape of a cross section in a stream depends on the point along the channel with reference to the plan geometry, the type channel, and the characteristics of the sediment forming and transported within the channel. The cross section in a bend is deeper at the **concave** (outer bank) side with a nearly vertical bank, and has a shelving bank as formed by the point bar on the **convex** side. The cross section will be more trapezoidal or rectangular in a crossing. These are shown in Figure 3.6. Cross section shape can be described by a number of variables. Some of these such as the **area**, **width**, and **maximum depth** are self explanatory. However, other commonly used parameters warrant some explanation. The **wetted perimeter (P)** refers to the length of the wetted cross section measured normal to the direction of flow. The **width-depth (w/d)** ratio is the channel width divided by the **average depth (d)** of the channel. The average depth is calculated by dividing the cross section area by the channel width. The **hydraulic radius (r)**, which is important in hydraulic computations is defined as the cross sectional area divided by the wetted perimeter. In wide channels with w/d greater than about 20 the hydraulic radius and the mean depth are approximately equal. The **conveyance**, or capacity of a channel is related to the area and hydraulic radius and is defined as $AR^{2/3}$.

Channel Bars. Channel bars are depositional features that occur within the channel. The size and location of bars are related to the sediment transport capacity and local geometry of the reach. The enlargement of a bar generally results in caving of the opposite banks in order to maintain conveyance of the discharge. The primary types of bars are point bars, middle bars, and alternate bars.

Point bars form on the inside (convex) bank of bends in a meandering stream. A typical point bar is shown in Figure 3.3. The size and shape of the point bar are determined by the characteristics of the flow. The development of a point bar is partially due to the flow separation zone caused by centrifugal forces in the bend, and secondary flow. **Middle bar** is the term given to areas of deposition lying within, but not connected to the banks. Figure 3.7 shows a typical middle bar on the Mississippi River. Middle bars tend to form in reaches where the crossing areas between bends are excessively long and occasionally in bends due to the development of chutes. **Alternate bars** are depositional features that are positioned successively down the river on opposite sides (Figure 3.8). Alternate bars generally occur in straight reaches and may be the precursor to a fully developed meander pattern.

3.1.3.3 Planform Geometry

Sinuosity is a commonly used parameter to describe the degree of meander activity in a stream. Sinuosity is defined as the ratio of the distance along the channel (channel length) to the distance along the valley (valley length). Think of sinuosity as the ratio of the distance the fish swims to the distance the crow flies. A perfectly straight channel would have a sinuosity of 1.0, while a channel with a sinuosity of 3.0 or more would be characterized by tortuous meanders. The

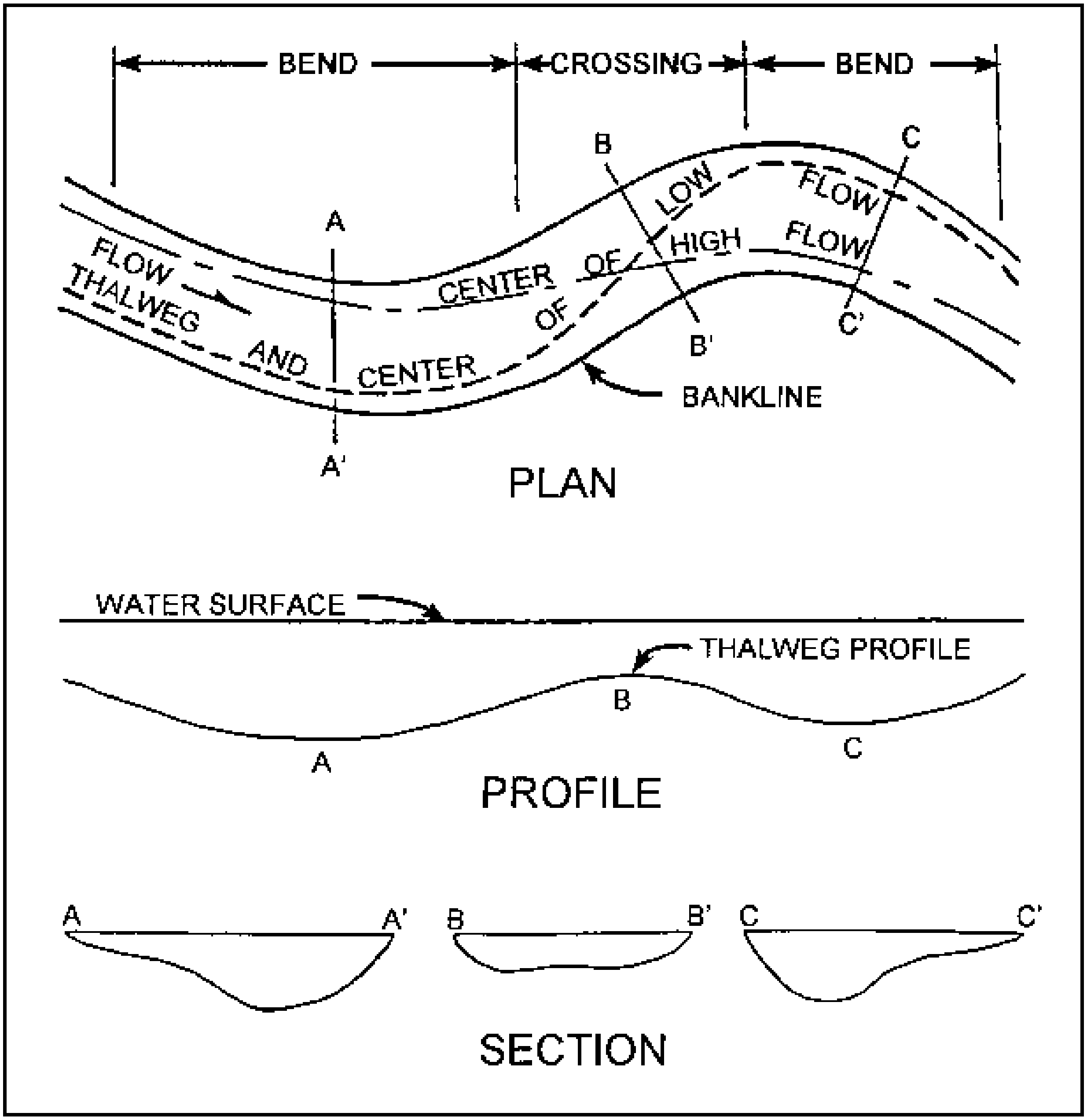


Figure 3.6 Typical Plan and Cross Sectional View of Pools and Crossings

Figure 3.7 Typical Middle Bar

Figure 3.8 Typical Alternate Bar Pattern

meander wave length (L) is twice the straight line distance between two consecutive points of similar condition (i.e. pools or crossings) in the channel as depicted in Figure 3.9. This is sometimes referred to as the axial meander wavelength to distinguish it from the channel length between inflection points which is also sometimes referred to as the meander wave length. The **meander amplitude (A)** is the width of the meander bends measured perpendicular to the valley or straight line axis (Figure 3.9). The ratio of the amplitude to meander wavelength is generally within the range 0.5 to 1.5. It should be noted that the meander amplitude and the width of the meander belt will probably be unequal. The meander belt of a stream is formed by and includes all the locations held by a stream during its development history. In many cases, this may include all portions of the present flood plain. Meander wave length and meander width are primarily dependent on the water and sediment discharge, but may also be modified by confines of the material in which the channel is formed. The effects of bank materials is shown by the irregularities found in the alignment of natural channels. If the material forming the banks was homogeneous over long distances, a sinusoidal alignment having a unique and uniform meander wavelength would be expected although this rarely occurs in nature.

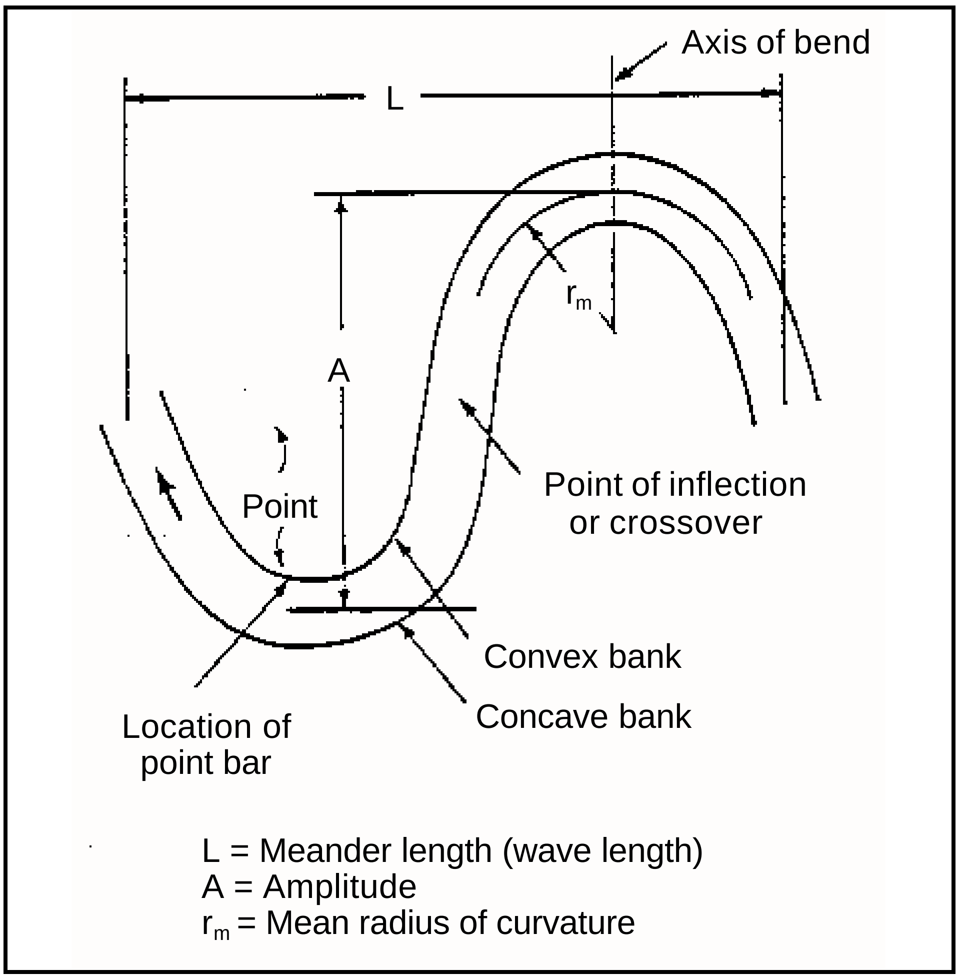


Figure 3.9 Definition Sketch for Channel Geometry (after Leopold *et al.*, 1964)

The **radius of curvature (r)** is the radius of the circle defining the curvature of an individual bend measured between adjacent inflection points (Figure 3.9). The **arc angle (è)** is the angle swept out by the radius of curvature between adjacent inflection points. The radius of curvature to width ratio (r/w) is a very useful parameter that is often used in the description and comparison of meander behavior, and in particular, bank erosion rates. The radius of curvature is dependent on the same factors as the meander wavelength and width. Meander bends generally develop a radius of curvature to width ratio (r/w) of 1.5 to 4.5, with the majority of bends falling in the 2 to 3 range.

3.1.3.4 Channel Slope

The slope (longitudinal profile) of a stream is one of the most significant parameters in the study and discussion of river behavior. Slope is one of the best indicators of the ability of the river to do work. Rivers with steep slopes are generally much more active with respect to bank erosion, bar building, sediment movement, etc., than lower slope channels.

Slope can be defined in a number of ways. If sufficient data exists, the water surface slope may be calculated using stage readings at gage locations along the channel. However, in many instances, particularly in small streams, gage information is non-existent. In these cases, the thalweg slope is generally calculated. Thalweg refers to the deepest point in a cross section. The thalweg slope not only provides a good expression of the energy of the stream, but also may aid in locating areas of scour and fill, geologic controls, and outcrops of non-erodible materials.

3.1.4 RELATIONSHIPS IN RIVERS

One interesting aspect of meandering rivers is the similarity in the proportion of planform characteristics. Various empirical relationships have been developed which relate radius of curvature and meander wavelength to channel width and discharge. Brice (1984) suggested that these similarities, regardless of size, account for the fact that the meandering planform is sensibly independent of scale. In other words, if scale is ignored, all meandering rivers tend to look alike in plan view. This fact provides us with a glimmer of hope that we might be able to develop relationships to help explain the behavior of complex river systems.

Investigation by Lane (1957) and Leopold and Wolman (1957) showed that the relationships between discharge and channel slope can define thresholds for indicating which rivers tend to be braided or meandering, as shown in Figures 3.10 and 3.11. Lane's relationship is somewhat more realistic because an intermediate range is included; however, both relationships are very similar in the variables used and the appearance of the graphs. Rivers that are near the threshold lines may exhibit segments that transitions between the two planforms. These relationships can be useful if the planform of a river is to be changed. For instance, a meandering river positioned at point ‘A’ in Figure 3.11 might be shifted to point ‘B’ if the slope is increased due to the construction of man-made cutoffs.

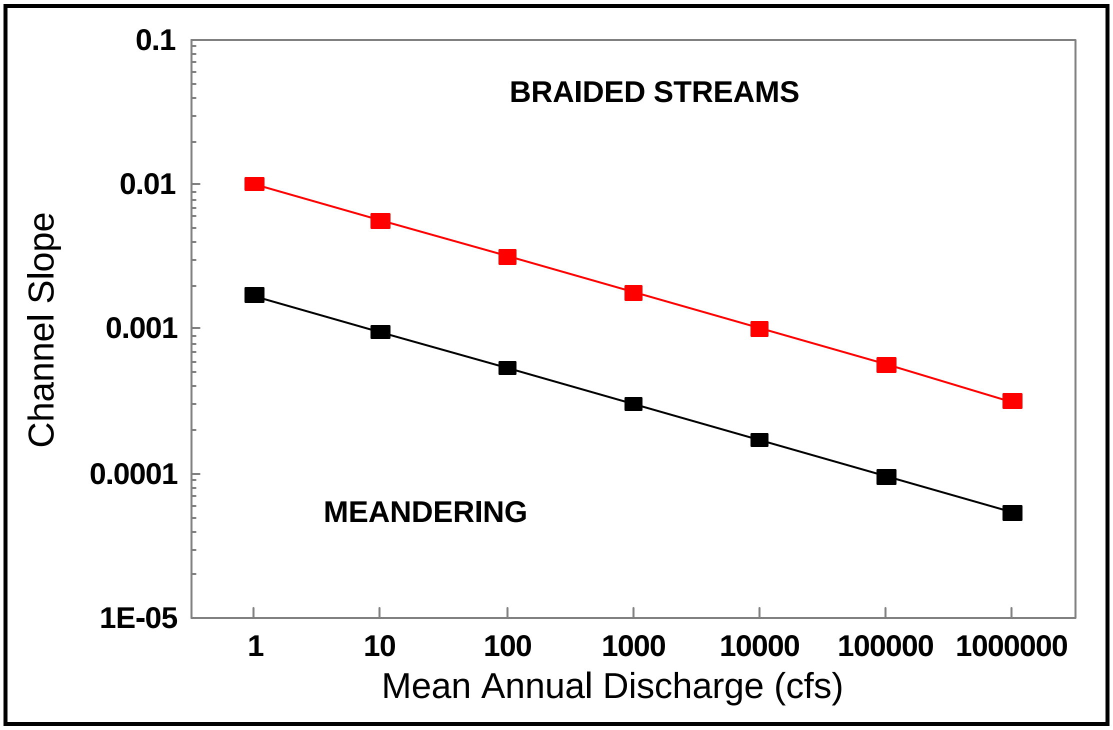


Figure 3.10 Lane's (1957) Relationship Between Channel Patterns, Channel Gradient, and Mean Discharge

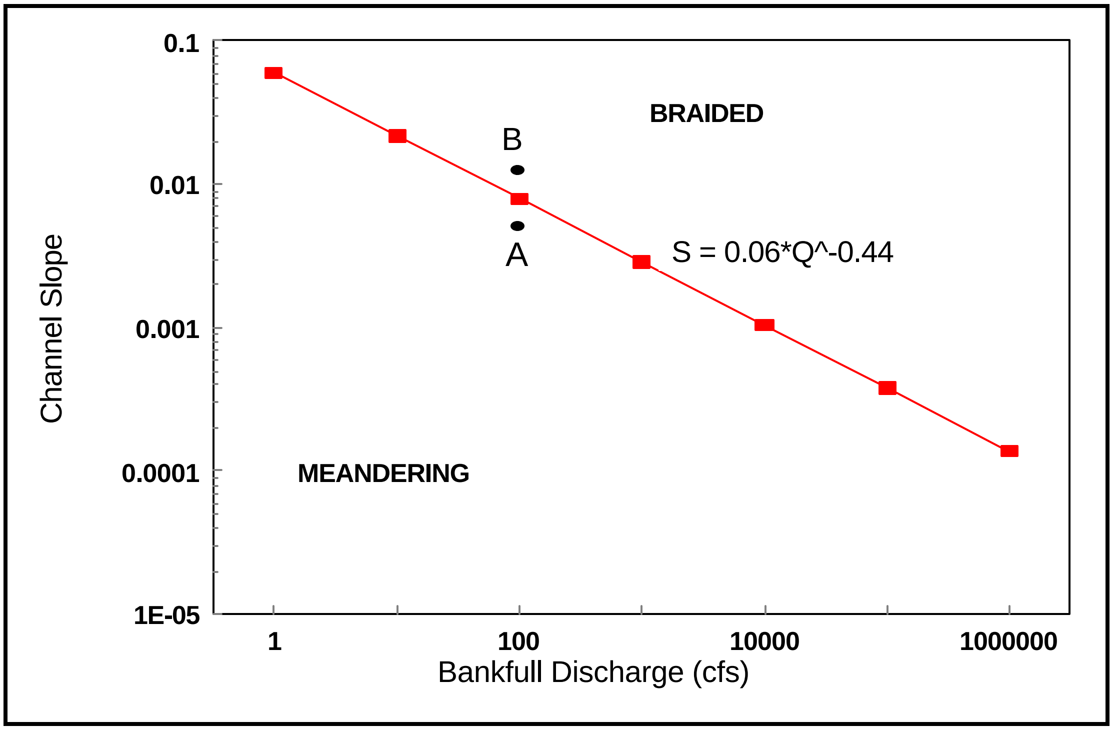


Figure 3.11 Leopold and Wolman's (1957) Relationship Between Channel Patterns, Channel Gradient, and Bankfull Discharge

Another set of empirical relationships is related to meander geometry. Leopold *et al*. (1964) reported the relationship between meander wave length (L) and channel width (w), meander amplitude (A) and channel width (w), and meander wave length (L) and bendway radius of curvature (R_c) as defined by Leopold and Wolman (1960). The relationships are:

$$L = 10.9\ w^{1.01}$$

$$A = 2.7\ w^{1.1}$$

$$L = 4.7\ R_c^{0.98}$$

Leopold *et al*. (1964) stated that the exponents for the relationships are approximately unity, and these relationships can be considered linear. Also, they pointed out that channel meander form is affected by the cohesiveness of the channel boundaries. Dury (1964) found that meander wave length is related to the mean annual flood (Q_{ma}):

$$L = 30\ Q_{ma}^{0.5}$$

Schumm (1960, 1977) investigated the effect of the percentage silt and clay (M) in the stream boundaries and reported the following relationship for meander wave length:

$$L = 1890\ Q_m^{0.34}\ M^{-0.74}$$

where Q_m is the average annual flow. The width to depth ratio (F) is also related to the percentage silt and clay:

$$F = 255\ M^{-1.08}$$

Channel slope (S) was found to be related to the mean annual discharge (Q_m) and percentage silt and clay:

$$S = 60\ M^{-0.38}\ Q_m^{-0.32}$$

Regime theory is an application of the idea that the width, depth, slope, and planform of a river are adjusted to a channel-forming discharge. In his review of the history of regime theory, Lane (1955) states that in 1895 Kennedy proposed the following relationship:

$$V = cD^m$$

in which V is the mean channel velocity, D is the channel depth, and c and m are constants developed for various channel locations. Much of the early work in developing regime relationships was conducted in the irrigation canals of India, and since the early 1900s, many relationships have been proposed.

Leopold and Maddock (1953) compiled a significant statistical data base using USGS gauging records and developed **hydraulic geometry** relationships for the width, depth, velocity, and other hydraulic characteristics for some streams in the United States. The hydraulic geometry relationships are of the same general form as Kennedy (1895):

$$W = a\,Q^{b}$$
$$D = c\,Q^{f}$$
$$V = k\,Q^{m}$$

in which W is channel width, Q is discharge, D is depth, and V is velocity.

All of the relationships presented, including the hydraulic geometry relationships, are strictly empirical, i.e., the relationships describe observed physical correlations. As conditions change from watershed to watershed, the relationships must be modified. For example, stream width for sandy banks would be expected to be different from clay banks. Schumm's relationship between width to depth ratio (F) and the weighted percent silt-clay in the channel perimeter (M) is an empirical relationship that describes this observation. If Schumm's relationship is correct, then is the hydraulic geometry relationship valid that predicts width (W) based only as a function of discharge? Both relationships can be valid for the data set used in developing the relationship.

An example of the improper use of empirical relationships was provided by Mark Twain in *Life on the Mississippi* (Clemens, 1944). In his wonderfully sarcastic manner, he describes Mississippi River cutoffs of which he had knowledge. Therefore, he developed an empirical relationship to predict the eventual length of the Mississippi River. He eloquently describes the modeling process:

> *Now, if I wanted to be one of those ponderous scientific people, and "let on" to prove what had occurred in the remote past by what had occurred in a given time in the recent past, or what will occur in the far future by what has occurred in late years, what an opportunity is here! Geology never had such a chance, nor such exact data to argue from! Nor "development of species," either! Glacial epochs are great things, but they are vague - vague. Please observe:*
>
> *In the space of 176 years, the Lower Mississippi has shortened itself 242 miles. That is an average of a trifle over one mile and a third per year. Therefore, any calm person, who is not blind or idiotic, can see that in the Old Oölitic Silurian Period, just a million years ago next November, the Lower Mississippi River was upwards of 1,300,000 miles long, and stuck out over the Gulf of Mexico like a fishing rod. And by the same token, any person can see that 742 years from now the Lower Mississippi will be only a mile and three-quarters long, and Cairo and New Orleans will have joined their streets together, and be plodding comfortably along under a single mayor and a mutual board of aldermen. There is something fascinating about science. One gets such wholesale returns of conjecture out of such a trifling investment of fact.*

The primary point of this delightful sarcasm is that we should not fall into the trap of attempting to plan a project based on "...wholesale returns of conjecture out of such a trifling investment of fact." Empirical relationships can be very useful. We cannot be certain that New Orleans and St. Louis will have a common Board of Aldermen on September 13, 2604; however we must be certain that the data from which the relationship was developed is valid for the project location, for the scale of the project, and that the relationship makes physical sense in application to the project.

3.1.5 CHANNEL CLASSIFICATION

Several primary methods of river classification are presented in the following paragraphs, and these methods can be related to fundamental variables and processes controlling rivers. One important classification is either alluvial or non-alluvial. An **alluvial** channel is free to adjust dimensions such as size, shape, pattern, and slope in response to change and flow through the channel. The bed and banks of an alluvial river are composed of material transported by the river under present flow conditions. Obviously, a **non-alluvial river** is not free to adjust. An example of a non-alluvial river is a bedrock controlled channel. In other conditions, such as in high mountain streams flowing in very coarse glacially deposited materials or significantly controlled by fallen timber would suggest a non-alluvial system.

Alluvial channels may also be classified as either perennial, intermittent, or ephemeral. A **perennial stream** is one which has flow at all times. An **intermittent stream** has the potential for continued flow, but at times the entire flow is absorbed by the bed material. This may be seasonal in nature. An **ephemeral stream** only has flow following a rainfall event. When carrying flow, intermittent and ephemeral streams both have characteristics very similar to perennial streams.

Another classification methodology by Schumm (1977) includes consideration of the type of sediment load being transported by the stream, the percentage of silt and clay in the channel bed and banks, and the stability of the channel. **Sediment load** refers to the type or size of material being transported by a stream. The total load can be divided into the **bed sediment load** and the **wash load**. The bed sediment load is composed of particles of a size found in appreciable quantities in the bed of the stream. The wash load is composed of those finer particles that are found in small quantities in the shifting portions of the bed. Frequently, the sediment load is divided into the **bedload**, those particles moving on or near the bed, and the **suspended load**, those particles moving in the water column. The size of particles moving as suspended load may include a portion of the bed sediment load, depending on the energy available for transport (Vanoni, 1977). For example, the suspended load frequently reported by U.S. Geological Survey publications usually includes a portion of the bed sediment load and all of the wash load. **Sediment discharge** is the rate at which the sediment load is being supplied or transported through a reach.

For purposes of this classification system, a stable channel complies with Mackin's definition of a graded stream. An unstable stream may be either **degrading** (eroding) or **aggrading** (depositing). In the context of the definition of a graded stream being in balance between sediment supplied and sediment transported, an aggrading stream has excess sediment supply and a degrading stream has a deficit of sediment supply.

Table 3.2 presents a summary of this classification system and describes the response of the river segment to instability and a description of the stable segment. It is very important to note that the work on which this classification was based was conducted in the midwestern U.S.; therefore, the classification system represents an interpretation of empirical data. Extrapolation of the classification beyond the database should be done cautiously.

Schumm and Meyer (1979) presented the channel classification shown in Figure 3.12, which is based on channel planform, sediment load, energy, and relative stability. As with any classification system, Figure 3.12 implies that river segments can be conveniently subdivided into clearly discernable groups. In reality, a continuum of channel types exists and the application of the classification system requires judgement.

Other stream classifications include those by Neill and Galay (1967) and by Rundquist (1975). These systems go well beyond a description of the channel, and include description of land use and vegetation in the basin, geology of the watershed, hydrology, channel bed and bank material, sediment concentration, channel pattern, and channel stability.

Rosgen (1994) presented a stream classification system similar to the Runquist (1975) system. A primary difference between the two systems is that planform and bed material character are combined into one code, improving the ease of use. Rosgen (1994) also included an entrenchment ratio, which is the ratio of the width of the flood-prone area to the surface width of the bankfull channel. Like Runquist (1975), Rosgen (1996) has also added valley type classification. Table 3.3 is a summary of delineative criteria for broad-level classification from Rosgen (1994). Each of the stream types can be associated with dominant bed material types as follows: Bedrock - 1, Boulder - 2, Cobble - 3, Gravel - 4, Sand - 5, and Silt/Clay - 6.

With some modifications to Figure 3.12, Figure 3.13 is a combination of some concepts of Schumm and Rosgen. Schumm's classification system was heavily dependent on his Midwestern experience, while Rosgen's experience began in steep mountain streams. In addition, Schumm's (1977) classification did not specifically include incised channels, which are included in Rosgen's (1994) F and G classes. Figure 3.13 includes C, D, DA, and E classes, and could be expanded to include all of Rosgen (1994) classes. The value of Figure 3.13 is to demonstrate that moving from class to class is a predictable response that manages energy, materials, and channel planform to reestablish a balance of sediment and water discharge with sediment and water supply.

3.2 CHANNEL EVOLUTION

The conceptual incised channel evolution model (CEM) has been of value in developing an understanding of watershed and channel dynamics, and in characterizing stable reaches of these channels. The sequence was originally used to describe the erosion evolution of Oaklimiter Creek, a tributary of Tippah River in northern Mississippi. Simon and Hupp (1987) have developed a similar model of channel evolution.

Table 3.2 Classification of Alluvial Channels (after Schumm, 1977)

Mode of sediment transport and type of channel	Channel sediment (M) (percent)	Bedload (percentage of total load)	Channel stability		
			Stable (graded stream)	Aggrading (excess sediment discharge)	Degrading (deficiency of sediment discharge)
Suspended load	>20	<3	Stable suspended-load channel. Width/depth ratio <10; sinuosity usually >2.0; gradient, relatively gentle	Depositing suspended load channel. Major deposition on banks cause narrowing of channel; initial streambed deposition minor	Eroding suspended-load channel. Streambed erosion predominant; initial channel widening minor
Mixed load	5-20	3-11	Stable mixed-load channel. Width/depth ratio >10, <40; sinuosity usually <2.0, >1.3; gradient moderate	Depositing mixed-load channel. Initial major deposition on banks followed by streambed deposition	Eroding mixed-load channel. Initial streambed erosion followed by channel widening
Bed load	<5	>11	Stable bed-load channel. Width/depth ratio >40; sinuosity usually <1.3; gradient, relatively steep	Depositing bed-load channel. Streambed deposition and island formation	Eroding bed-load channel. Little streambed erosion; channel widening predominant

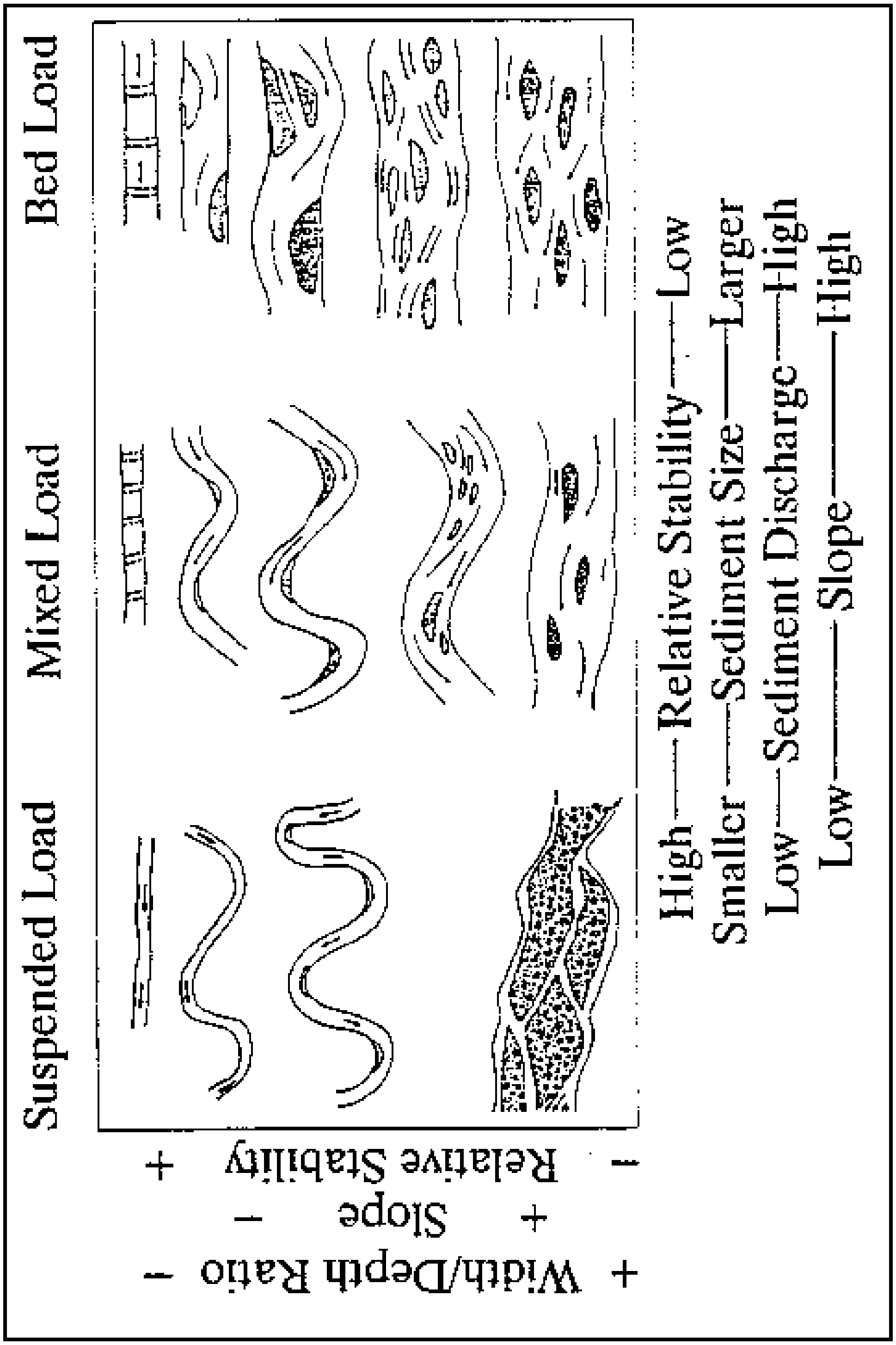


Figure 3.12 Channel Classification Based on Pattern and Type of Sediment Load (after Schumm, 1981)

Table 3.3 Summary of Delineative Criteria for Broad-level Classification (Rosgen, 1994)

Stream Type	Entrench. Ratio	W/D Ratio	Sinuosity	Slope	Meander Belt/ Bankfull Width	Dominant Bed Material*
Aa+	<1.4	<12	1.0 - 1.1	> 0.10	1.0 - 3.0	1,2,3,4,5,6
A	<1.4	<12	1.0 - 1.2	0.04 - 0.10	1.0 - 3.0	1,2,3,4,5,6
B	1.4 - 2.2	>12	>1.2	0.02 - 0.039	2.0 - 8.0	1,2,3,4,5,6
C	>2.2	>12	>1.4	< 0.02	4.0 - 20	1,2,3,4,5,6
D	na	>40	na	< 0.04	1.0 - 2.0	3,4,5,6
DA	>4.0	<40	variable	< 0.005	na	4,5,6
E	>2.2	<12	>1.5	< 0.02	20 - 40	3,4,5,6
F	<1.4	>12	>1.4	< 0.02	2.0 - 10	1,2,3,4,5,6
G	<1.4	<12	>1.4	0.02 - 0.039	2.0 - 8.0	1,2,3,4,5,6

*Dominant Bed Material Key

1 - Bedrock
2 - Boulder
3 - Cobble
4 - Gravel
5 - Sand
6 - Silt/Clay

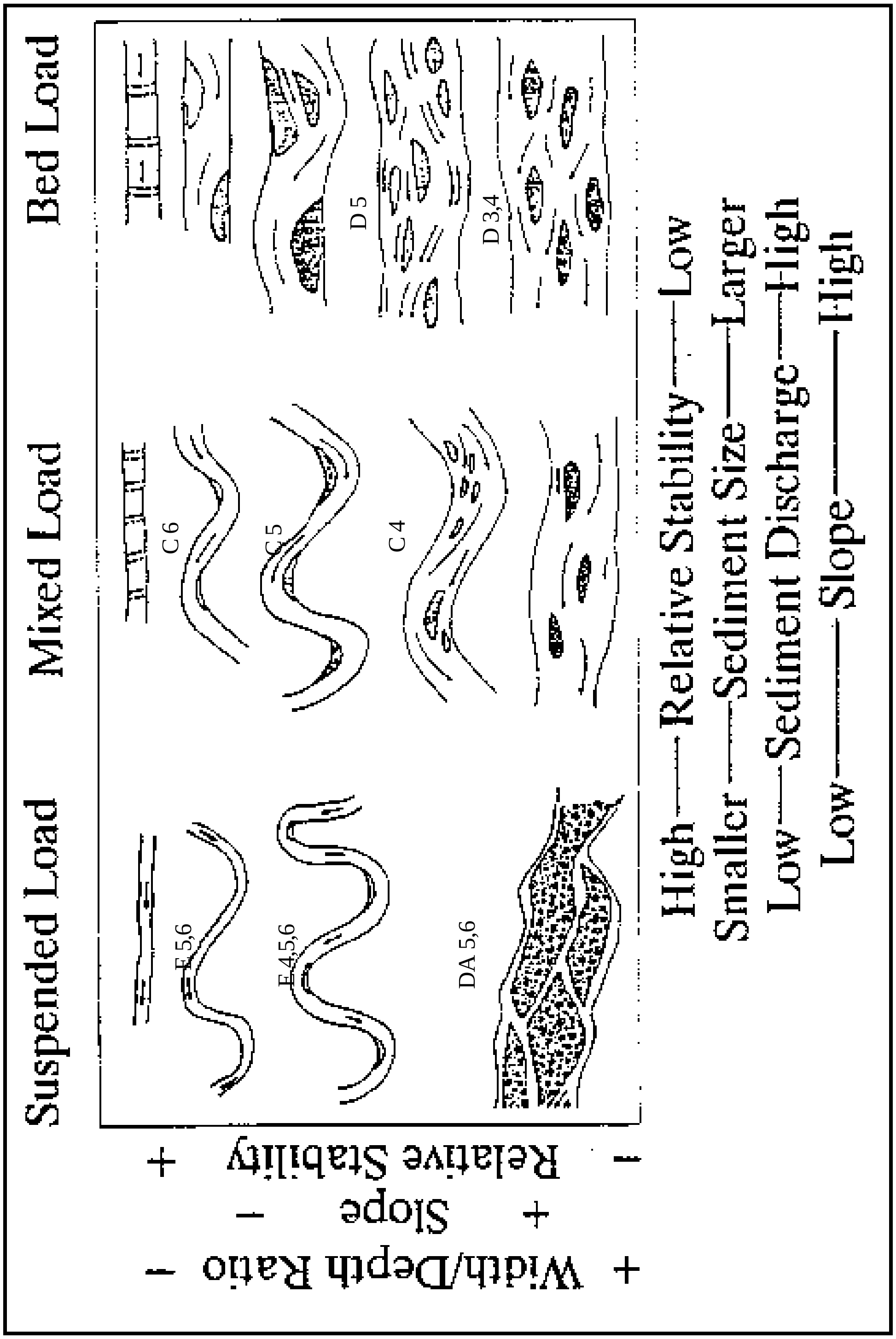


Figure 3.13 Channel Classification Combining Aspects of Schumm (1981) and Rosgen (1994)

Location-for-time substitution was used to generate a five-reach type, incised channel evolution sequence for stream of the Yazoo Basin (Schumm *et al*., 1984), as shown in Figure 3.14. In each reach of an idealized channel, Types I and V occur in series and, at a given location, will occur in the channel through time. The channel evolution model describes the systematic response of a channel to base level lowering, and encompasses conditions that range from disequilibrium to a new state of dynamic equilibrium. The following paragraphs characterize the conceptual types. It should be recognized that these categories are only conceptual and variation may be encountered in the field.

Type I reaches are characterized by: a sediment transport capacity that exceeds sediment supply, bank height (h) that is less than the critical bank height (hc), a U-shaped cross section, small precursor knickpoints in the bed of the channel providing that the bed material is sufficiently cohesive, and little or no bed material deposited. Width-depth ratios at bankfull stage are highly variable.

Type II reaches are located immediately downstream of the primary knickpoint and are characterized by: a sediment transport capacity that exceeds sediment supply, a bank height that is greater than the critical bank height ($h > hc$), little or no bed sediment deposits, a lower bed slope than the Type I reach, and a lower width-depth ratio value than the Type I reach because the depth has increased but the banks are not failing.

Type III reaches are located downstream of Type II reaches and are characterized by: a sediment transport capacity that is highly variable with respect to the sediment supply, a bank height that is greater than the critical bank height ($h > hc$), erosion that is due primarily to slab failure (Bradford and Piest, 1980), bank loss rates that are at a maximum, bed sediment accumulation that is generally less than two feet, but can locally be greater due to local erosion sources, and channel depth that is somewhat less than in Type II. The channel is widening due to bank failure.

Type IV reaches are downstream of Type III reaches and are characterized by: a sediment supply that exceeds sediment transport capacity resulting in aggradation of the channel bed, a bank height that approaches the critical bank height with a rate of bank failure lower than Type III reaches, a nearly trapezoidal cross-section shape, and a width-depth ratio higher than the Type II reaches. The Type IV reach is aggradational and has a reduced bank height. Bank failure has increased channel width, and in some reaches the beginnings of berms along the margins of an effective discharge channel can be observed. These berms are the initiation of natural levee deposits that form in aggraded reaches that were over-widened during earlier degradational phases. Bradford and Piest (1980) observed that in the later phases of evolution, the mode of bank failure changes from circular arc to slab-type failures.

Type V reaches are located downstream of Type IV reaches and are characterized by: a dynamic balance between sediment transport capacity and sediment supply for the effective discharge channel, a bank height that is less than the critical bank height for the existing bank angle, colonization by riparian vegetation, an accumulated bed sediment depth that generally exceeds 3 feet, a width-depth ratio that exceeds the Type IV reach, and generally a compound channel formed within a newly formed floodplain. The channel is in dynamic equilibrium. Bank angles have been reduced

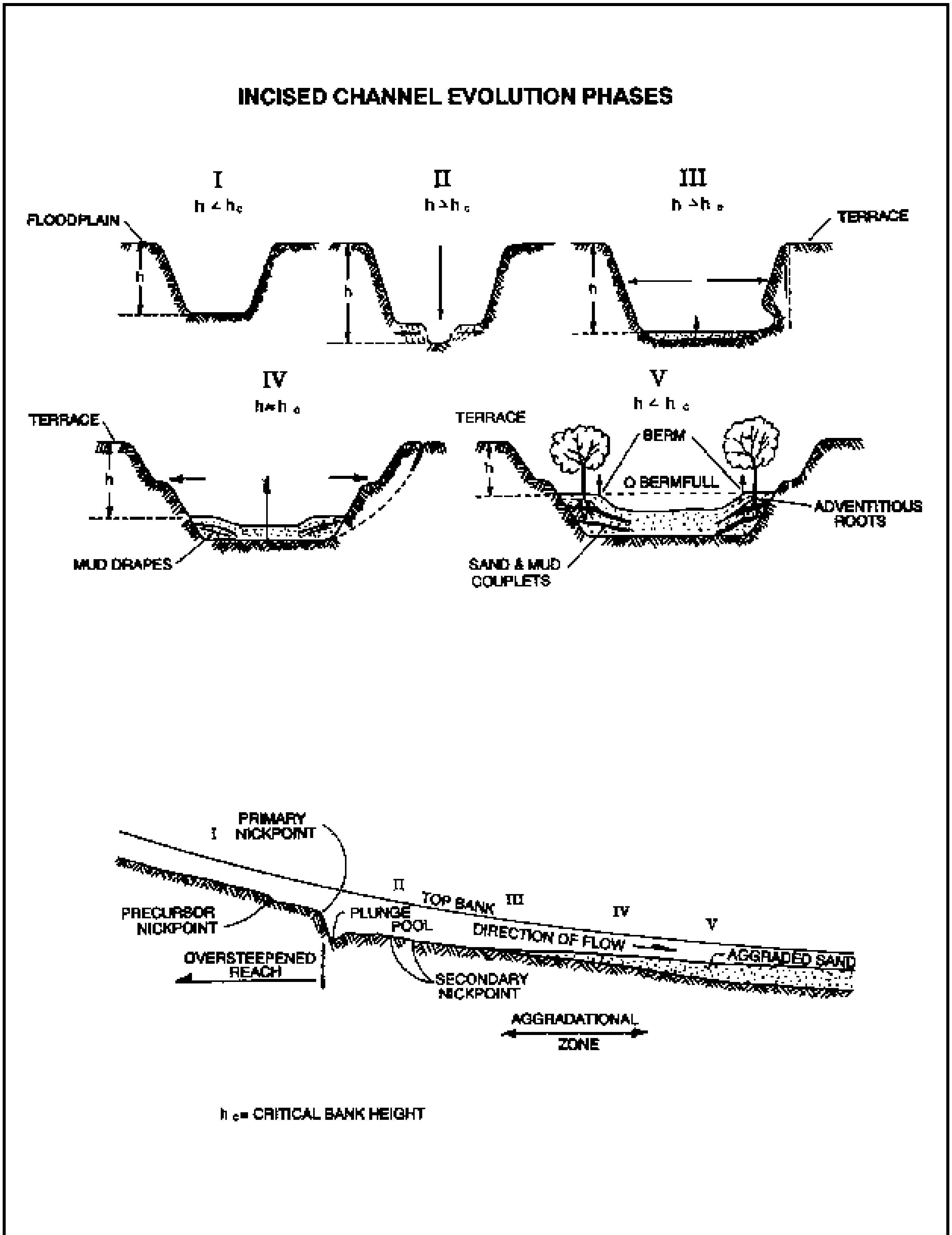


Figure 3.14 Incised Channel Evolution Sequence (after Schumm *et al*., 1984)

by accumulation of failed bank materials at the toe of the slope and by accumulation of berm materials.

The sequence of channel evolution is based on the assumption that the observed changes in channel morphology are due to the passage of time in response to a single base level lowering without changes in the upstream land use and sediment supply from the watershed. Application of the sequence assumes that the materials forming the channel perimeter are erodible and all degrees of the channel adjustment are possible. The sequence is applicable only in a system context, and local erosion such as in bends or caused by deflection of flow by debris may cause difficulty in application of the sequence.

The primary value of the sequence is to determine the evolutionary state of the channel from a field reconnaissance. The morphometric characteristics of the channel reach types can also be correlated with hydraulic, geotechnical, and sediment transport parameters (Harvey and Watson, 1986; Watson *et al.*, 1988). The evolution sequence provides an understanding that reaches of a stream may differ in appearance, but channel form in one reach is associated with other reaches by an evolving process. Form, process, and time relate dissimilar reaches of the stream.

The USACE (1990) used the channel evolution sequence in developing regional stability curves relating the bed slope of Type V reaches as a function of the measured drainage area. Quasi-equilibrium, Type V reaches were determined by field reconnaissance of knowledgeable personnel. Figure 3.15 is an example of the empirical bed slope and drainage area relationship for Hickahala Creek, in northern Mississippi. The 95% confidence intervals of the regression line are shown. The slope-area curve is an example of an empirical relationships that does not explicitly include the primary factors of water and sediment discharge, sediment load, hydraulic roughness, and channel morphology.

Watson *et al*. (1995b) stated that stream classification is an essential element in transferring knowledge and experience pertaining to channel design from location to location. A computer program was developed to record a comprehensive data set for a watershed and for channel sites, and to present alternative classification of each based on three classification systems: Schumm (1977), Rosgen (1994), or Montgomery and Buffington (1993). A goal of the program was to develop understanding between groups who are most familiar with only on or two of the classification systems compared. Watson *et al*. (1995a) found that the improvement in stability of the incised reaches has resulted in lower channel slope and sediment yield. Use of a previous slope-area curve data based on generally less stable channel characteristics, results in the design of a channel that would be stable at higher sediment yields than are now present in the more stable DEC streams. The slope-area curve must be constantly updated, or a design method that specifically includes sediment yield should be used.

3.3 QUANTIFICATION OF THE EVOLUTIONARY SEQUENCE

The parameters of the Channel Evolution Model, Section 3.2, are difficult to quantify and to incorporate in design guidance. The parameters can be compressed into two dimensionless

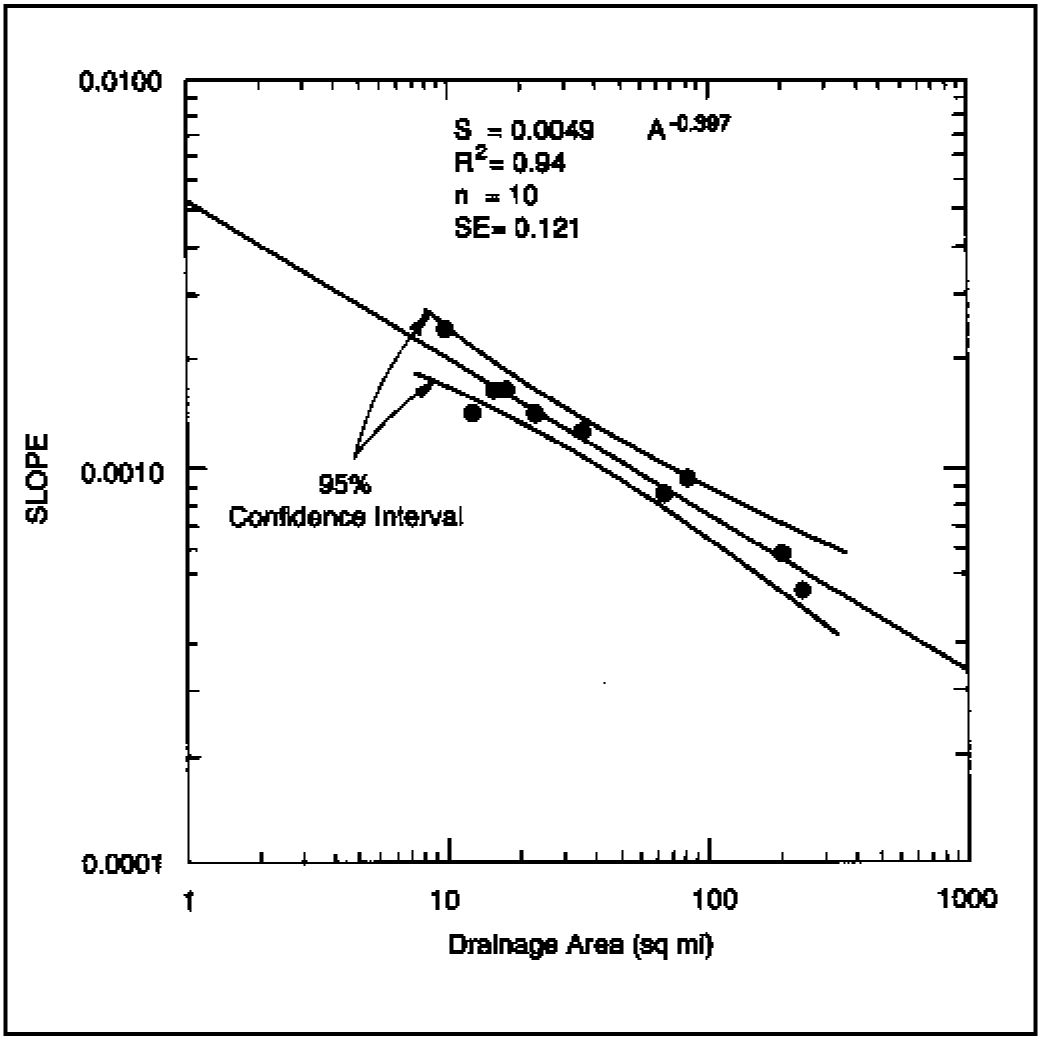


Figure 3.15 Hickahala Creek Watershed, Slope-drainage Area Relationship

stability numbers: Ng is a measure of bank stability and Nh is a measure of sediment continuity. For a channel to be stable, sediment continuity and bank stability are essential.

Ng is defined as the ratio between the existing bank height and angle (h) and the critical bank height at the same bank angle (hc). Bank stability is attained when Ng is less than unity (Ng < 1). Therefore, Ng provides a rational basis for evaluating the requirements for bank stabilization and for evaluating the consequences of further bed degradation.

The hydraulic stability number, Nh, is defined as the ratio between the desired sediment supply and the actual sediment transport capacity. Sediment continuity yields Nh = 1.0. It is important to note that the definition of Nh includes sediment transport and supply, which is in contrast to most channel design procedures. Hydraulic stability in the channel is attained when Nh = 1. If Nh is < 1 the channel will

aggrade, and if Nh is > 1 it will degrade. Since sediment supply to a channel can change through time, it is prudent to design rehabilitation measures that will allow for the fluctuations in sediment supply.

In combination, Ng and Nh provide a set of design criteria that define both bank and hydraulic stability in the channel. Grade-control structures constructed in the channel should induce upstream deposition of sediment in the bed of the channel. This emulates the natural evolution of the channel. Reduction in the sediment transport capacity as a result of slope reduction permits deposition of sediment. This reduces the bank height of the channel. Continued bank erosion will occur only if the failed bank materials are removed by fluvial processes. The aggradation upstream of the grade-control structure eventually will result in increasing bank stability.

The dimensionless stability numbers, Ng and Nh, can be related to the channel evolution modes, as shown in Figure 3.16. As the channel evolves from a state of disequilibrium to a state of dynamic equilibrium through the five reach types of the Oaklimiter Sequence, the channel condition will progress through the four stability diagram quadrants in a counter-clockwise direction. Rehabilitation of the channel should attempt to omit as many of the quadrants as possible to reduce the amount of channel deepening and widening.

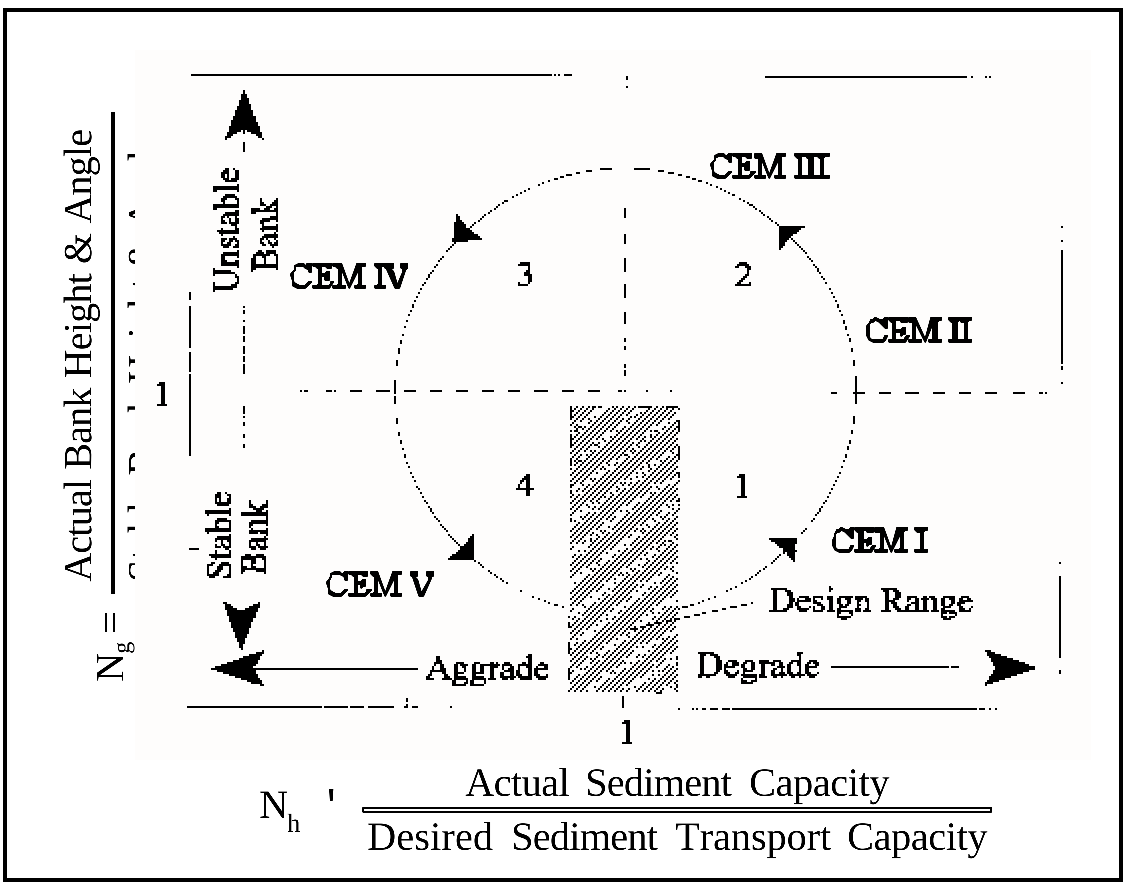


Figure 3.16 Comparison of the Channel Evolution Sequence and the Channel Stability Diagram

Each quadrant of the stability diagram is characterized by geotechnical and hydraulic stability number pairs, and stream reaches that plot in each quadrant have common characteristics with respect to stability, flood control, and measures that may be implemented to achieve a project goal.

Quadrant 2 (Ng > 1, Nh > 1) streams are severely unstable; the channel bed is degrading and channel banks are geotechnically unstable. Grade control must be used to reduce bed slope, transport capacity, and Nh. Both flood control and bank stability must be considered when determining the height to which grade control should be constructed. A series of grade control structures can reduce bank height enough to stabilize the banks, but a combination of grade control and bank sloping may better resolve flood control while meeting stability objectives. Quadrant 1 (Ng < 1, Nh > 1) is not as severe as Quadrant 2; the channel bed may be degrading or may be incipiently degradational, but the channel bank is not yet geotechnically unstable. Bank erosion is occurring only locally and bank stabilization measures such as riprap, dikes, or vegetation could be applied. However, local stabilization would not be successful if bed degradation continued, moving to Quadrant 2, and destabilized the channel stabilization measures. If flood control is a project goal, almost any channelization measure or construction of levees would increase the Nh instability, shifting the value to the right and increasing the opportunity to make Ng > 1. Flow control using a reservoir can address flood control and improve stability if the new flow duration curve reduces cumulative sediment transport; however, changing the flow duration curve and reducing the available sediment supply are potentially destabilizing. Each of these factors should be considered in projects involving Quadrant 1 channels. Bed stabilization through the use of a grade control structure or bed stabilization element may be desirable.

Quadrant 3 (Ng > 1, Nh < 1) has a severe and dynamic problem with gravity driven bank failure, but without continued bed degradation. Bank sloping could be effective without grade control emplacement, but usually both measures should be considered. Local bank stabilization measures in either Quadrant 2 or 3 are unlikely to be successful. Flow control in these two quadrants could be beneficial, but must be considered in the context of extreme reach instability and grade control is likely to be required.

Quadrant 4 (Ng < 1, Nh < 1) is characterized by general aggradation. Local bank stabilization measures will be effective. As Nh decreases in this quadrant, the potential for channel aggradation-related flood control problems increases.

The desirable range for long-term channel stability is for Ng to be less than one, and for Nh to be approximately one (Ng < 1, Nh • 1). If flood capacity is not sufficient as Ng approaches 1.0, levees or a compound channel should be considered.

The USACE (1990) used the channel stability diagram in discussions of Nelson, Beards, Catheys, and James Wolf Creeks stability, as shown in Figure 3.17. Figure 3.18 depicts the change in plotting positions of the result of channel stabilization measures that move two streams from degradation to aggradational (Stream A), and from degradational and unstable banks to aggradational and stable banks (Stream B). The proper characteristics for long-term stability are neither aggrading nor degrading, with stable banks.

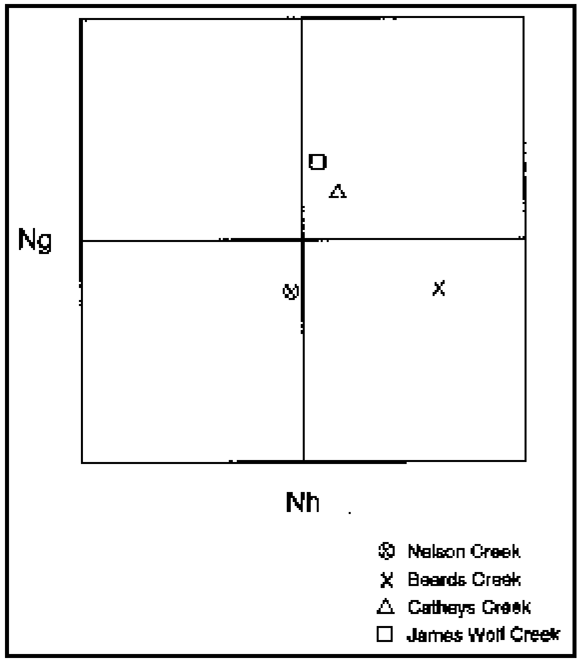


Figure 3.17 Sub-watershed Channels of Hickahala Creek Watershed Plotted on an Ng/Nh Diagram (after USACE, 1990)

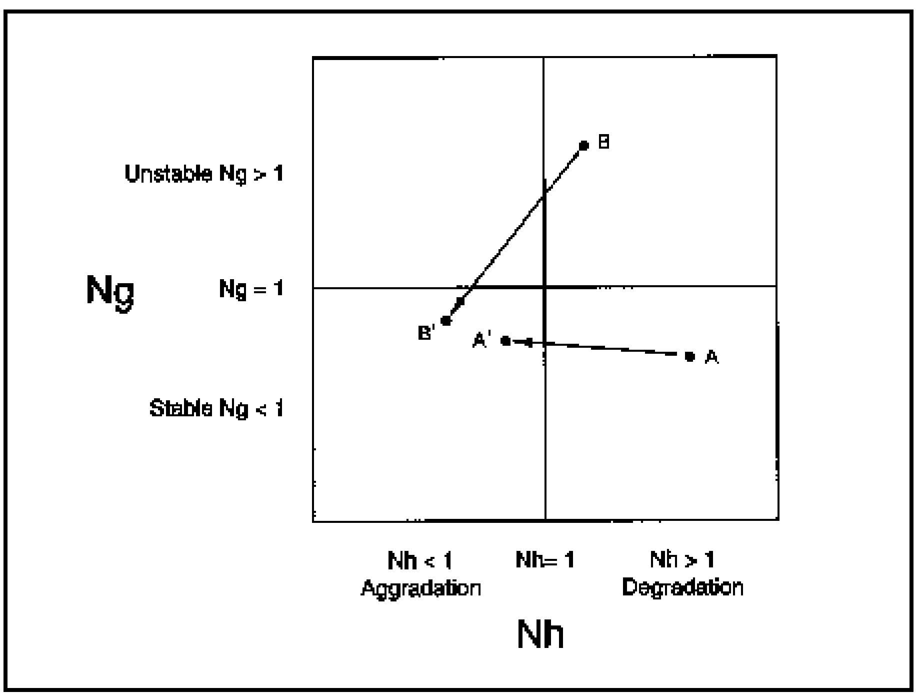


Figure 3.18 Dimensionless Stability Number Diagram for Stabilization Measures on Two Hypothetical Streams

3.4 CHANNEL STABILITY CONCEPTS

Streambank protection measures often fail, not as the result of inadequate structural design, but rather because of the failure of the designer to incorporate the existing and future channel morphology into the design. For this reason, it is important for the designer to have some general understanding of stream processes to insure that the selected stabilization measures will work in harmony with the existing and future river conditions. This section describes the basic concepts of channel stability. This will allow the designer to assess whether the erosion at a particular site is due to local instability processes or is the result of some system-wide instability problems that may be affecting the entire watershed.

3.4.1 THE STABLE CHANNEL

The concept of a stable river is one that has generated controversy between engineers, scientists, landowners, and politicians for many years. An individual's definition of stability is often subjectively based on past experiences or project objectives. To the navigation engineer, a stable river might be one that maintains adequate depths and alignment for safe navigation. The flood control engineer on the other hand is more concerned with the channel maintaining the ability to pass the design flood, while to the local landowner a stable river is one that does not erode the bankline. Therefore, bank erosion would not be an acceptable component of these groups' definition of a stable river. Geomorphologists and biologists, on the other hand, might maintain that bank erosion is simply part of the natural meandering process of stable rivers and would be perfectly acceptable in their definition of a stable river. Consequently, there is no universally accepted definition of a stable river. However, some manner of defining stability is needed before the concept of instability can be discussed. Therefore, the following paragraphs will attempt to establish a definition of a stable river to be used for this manual.

River behavior may be influenced by a number of factors. Schumm (1977) identified these as independent and dependent variables. Independent variables may be thought of as the basin inputs or constraints that cause a change in the channel morphology. Independent variables include: basin geology, hydrology (discharge of water and sediment), valley dimensions (slope, width, depth), vegetation (type and density), and climate. Dependent variables include: channel slope, depth, width, and planform.

A channel that has adjusted its dependent variables to accommodate the basin inputs (independent variables) is said to be stable. Mackin (1948) gave the following definition of a **graded stream**:

> *A graded stream is one in which, over a period of years, slope is delicately adjusted to provide, with available discharge and with prevailing channel characteristics, just the velocity required for the transportation of the load supplied from the drainage basin. The graded stream is a system in equilibrium.*

Mackin did not say that a stream in equilibrium is unchanging and static. A more commonly used term today for this type of stability is **dynamic equilibrium**. A stream in dynamic equilibrium has adjusted its width, depth and slope such that the channel is neither aggrading nor degrading. However, change may be

occurring in the stream bank, erosion may result, and bank stabilization may be necessary, even on the banks of a stream in dynamic equilibrium.

The equilibrium concept of streams discussed above can also be described by various qualitative relationships. One of the most widely used relationships is the one proposed by Lane (1955) which states that:

$$QS \propto Q_s D_{50}$$

where Q is the water discharge, S is the slope, Q_s is the bed material load, and D_{50} is the median size of the bed material. This relationship, commonly referred to as Lane's Balance, is illustrated in Figure 3.19. Mackin's concept of adjustment to changes in the controlling variables is easily illustrated by Lane's balance (Figure 3.19) which shows that a change in any of the four variables will cause a change in the others such that equilibrium is restored. When a channel is in equilibrium, it will have adjusted these four variables such that the sediment being transported into the reach is transported out, without significant deposition of sediment in the bed (aggradation), or excessive bed scour (degradation). It should be noted that by this definition of stability, a channel is free to migrate laterally by eroding one of its banks and accreting the one opposite at a similar rate.

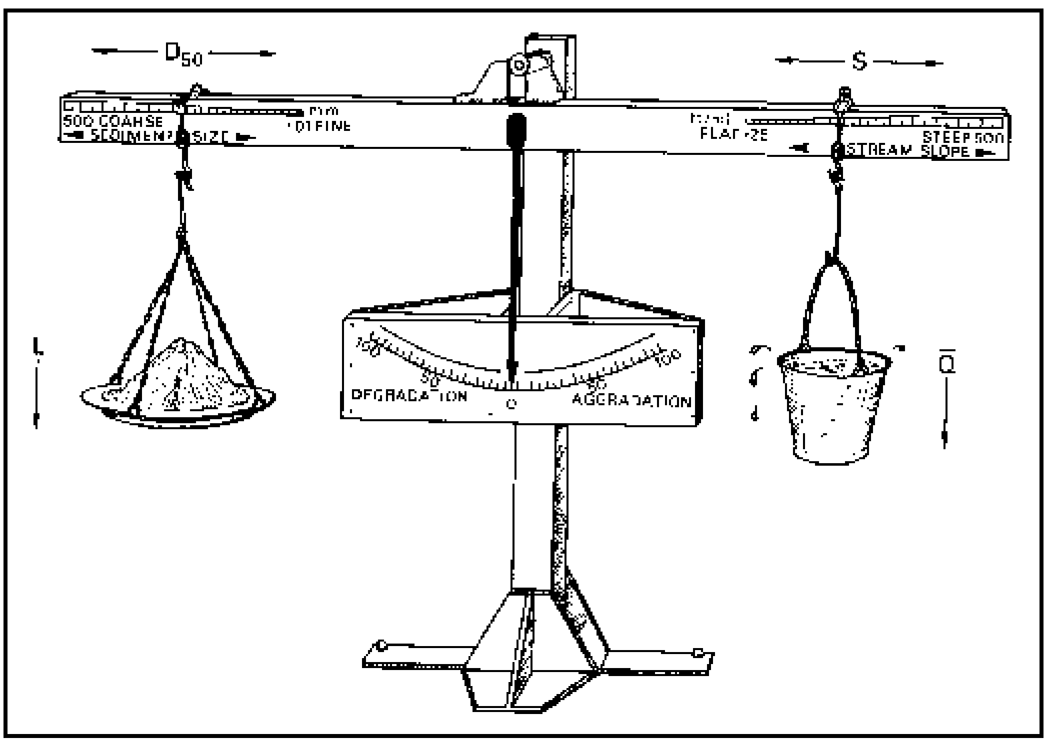


Figure 3.19 Lane’s Balance (after E. W. Lane, from W. Borland)

Meandering can be thought of as nature's way of adjusting its energy (slope) to the variable inputs of water and sediment. Cutoffs (oxbow lakes) and abandoned courses in the floodplain attest to the

dynamic behavior of rivers. Oftentimes the engineer or scientist draws the erroneous conclusion that a dis-equilibrium condition exist because natural cutoffs are occurring. However, this type of dynamic behavior is quite common in rivers that are in a state of dynamic equilibrium. In this situation, as natural cutoffs occur, the river may be obtaining additional length elsewhere through meandering, with the net result being that the overall reach length, and therefore slope, remains unchanged.

In summary, a stable river, from a geomorphic perspective, is one that has adjusted its width, depth, and slope such that there is no significant aggradation or degradation of the stream bed or significant planform changes (meandering to braided, etc.) within the engineering time frame (generally less than about 50 years). By this definition, a stable river is not in a static condition, but rather is in a state of dynamic equilibrium where it is free to adjust laterally through bank erosion and bar building. This geomorphic definition of stability (dynamic equilibrium) is developed here to establish a reference point for the discussion of system and local instability in the following sections.

3.4.2 SYSTEM INSTABILITY

The equilibrium of a river system can be disrupted by various factors. Once this occurs the channel will attempt to re-gain equilibrium by making adjustments in the dependent variables. These adjustments are generally reflected in channel aggradation (increasing bed elevation), degradation (decreasing bed elevation), or changes in planform characteristics (meander wavelength, sinuosity, etc.). Depending upon the magnitude of the change and the basin characteristics (bed and bank materials, hydrology, geologic or man-made controls, sediments sources, etc.) these adjustments can propagate throughout the entire watershed and even into neighboring systems. For this reason, the disruption of the equilibrium condition will be referred to as system instability.

As defined above system instability is a broad term describing the dis-equilibrium condition in a watershed. System instability may be evidenced by channel aggradation, degradation, or planform changes. This manual does not attempt to provide a complete discussion of all aspects of channel response, but rather, focuses primarily on the degradational and planform processes because these have the most significant impact on bank stability. For a more complete discussion of channel processes, the reader is referred to Simons and Sentürk (1992), Schumm (1972), Richards (1982), Knighton (1984), and Thorne *et al*. (1997).

Before the specific causes are addressed, a brief discussion of the consequences of system instability is necessary. The consequences of system instability can generally be discussed in terms of two components: (1) hydraulic consequences, and (2) geotechnical consequences. The consequences of system instability are illustrated in Figure 3.20. The hydraulic consequences of system instability are usually reflected in increased energy (discharge and slope) which result in excessive scour and erosion of the bed and banks. This erosion endangers bridges, buildings, roads, and other infrastructure, undermines pipeline and utility crossings, results in the loss of lands

(a) Bed and Bank Instability

(b) Formation of Gullies in Floodplain

Figure 3.20 Consequences of System Instability

(c) Damage to Infrastructure

(d) Excessive Sediment Deposition in Lower Reaches of Watershed

Figure 3.20 (cont.) Consequences of System Instability

adjacent to the stream, and generates a significant amount of sediment that is ultimately deposited downstream in navigation and flood control channels. The geotechnical consequences of system instability are a direct function of the hydraulic consequences of bed lowering. As degradation proceeds through a system, the channel bank heights and angles are increased, which reduces the bank stability with respect to mass failures under gravity. If degradation continues, eventually the banks become unstable and fail. Bank failures may then no longer be localized in the bendways, but rather may also be occurring along both banks in straight reaches on a system-wide basis. When this occurs, conventional bank stabilization measures are generally not suitable, and a more comprehensive treatment plan involving grade control or flow control dams, diversion structures, etc., is required.

3.4.2.1 Causes of System Instability

The stability of a channel system can be affected by a number of natural or man-induced factors. Natural geologic processes obviously cause dramatic changes but these changes generally occur over thousands or perhaps millions of years and, therefore, are not often a direct concern to the individual trying to stabilize a streambank. However, channel systems are significantly impacted within the engineering time span by the natural forces of earthquakes or volcanic eruptions. Although these phenomenon may have catastrophic consequences and receive considerable media attention, the most commonly encountered system instability problems can generally be attributed, at least in part, to man's activities.

Any time one or more of the controlling variables (runoff, sediment loads, sediment size, channel slope, etc.) in a watershed are altered there is a potential for inducing system instability. The particular system response will reflect the magnitude of change and the existing morphological sensitivity of the system. Therefore, each system is unique and there is no standard response that applies to all situations. With this in mind, it is not practical to attempt to discuss all the possible scenarios of channel response. Rather, the aim of this discussion is to present some of the more common factors causing system instability, and to illustrate how a particular channel response might be anticipated using the stability concepts discussed earlier.

A list and brief discussion of some of the more common causes of system instability are presented in the following sections. For this discussion the causes have been grouped into three categories: (1) downstream factors, (2) upstream factors, and (3) basin-wide factors. Following this, a brief discussion is presented concerning complex response and the complications involved when a system is subjected to multiple factors.

Downstream Factors. The stability of a channel system can be significantly affected by a downstream **base level** lowering. Base level refers to the downstream controlling water surface or bed elevation for a stream. One of the most common causes of base level lowering is the implementation of cutoffs or channelization as part of channel improvement projects (Figure 3.21). As indicated by Lane's relation (Figure 3.19), the increased slope must be offset by one of the other variables. Consequently, there is an imbalance between the sediment transport capacity and supply. If the discharge and bed material are assumed to remain constant (which may not always be the case), then the channel must adjust to the

increased slope (i.e., sediment transport capacity) by increasing its bed material load. This increased sediment load will be derived from the bed and banks of the channel in the form of channel degradation and bank erosion. As the bed continues to degrade, the zone of increased slope will migrate upstream and the increased bed material load is transmitted downstream to drive aggradational instability.

Figure 3.21 Channelized Stream and Abandoned Old Channel

The manner in which degradation migrates through a channel system is a very complex process. Before this process is discussed some of the relevant terminology must first be addressed. The following definition of terms is based on the terminology used by Schumm *et al*. (1984). Channel degradation simply refers to the lowering of the channel bed. Field indicators of degradation occur in the form of knickpoints or knickzones. A **knickpoint** is a location on the thalweg of an abrupt change of elevation and slope (Figure 3.22). This may also be visualized as a waterfall or vertical discontinuity in the stream bed. A steep reach of channel representing the headward migrating zone is referred to as a **knickzone** (Figure 3.23). A knickzone is often composed of a series of small knickpoints. Knickpoints and knickzones are often referred to as **headcuts**. While headcut is a commonly used term, it does generate some confusion because it is also used as a description of the headward migration process of degradation. To avoid this confusion the field indicators of degradation (knickpoints and knickzones) will not be referred to as headcuts. Rather, a headcut (or headcutting) is defined as a headward migrating zone of degradation. This headcutting may occur with or without the formation of knickpoints or knickzones which are purely a function of the materials encountered.

Figure 3.22 Knickpoint in a Degrading Channel

Figure 3.23 Knickzone in a Degrading Channel

Once headcutting is initiated it may proceed rapidly through the system. The rate of headward advance is a direct function of the materials encountered in the bed and also the basin hydrology. If the channel bed is composed primarily of non-cohesive sands and silts, then no knickpoints or knickzones will form and headcutting will work upstream by parallel lowering of the bed. However, if consolidated materials such as clays, sandstones, or other resistant materials occur in the channel bed, then knickpoints or knickzones will form as degradation encounters these resistant layers. When this occurs the headward migration rate may slow considerably. Therefore, the dominant factor affecting the headward migration rate is the relative resistance to erosion of the bed materials, and to a lesser degree the discharge in the stream.

As degradation migrates upstream it is not restricted to the main stem channel. When headcutting passes tributary junctions it lowers the base level of these streams. This initiates the degradation process for the tributaries. The localized increased slope at the confluence produces an excess sediment transport capacity that results in degradation of the stream bed. This process can continue upstream rejuvenating other tributaries until the entire basin has been affected by the downstream base level lowering.

Upstream Factors. System instability is often initiated by upstream alterations in the basin. This may result from a change in any of the controlling variables, but is most commonly associated with modifications to the incoming discharges of water and sediment. Looking at Lane's balance (Figure 3.19) it can be seen that either an increase in the water discharge or a decrease in the sediment load can initiate channel degradation. These factors are often altered by dams or channel diversions. A brief discussion of the effects of these features on the channel stability follows.

Channel response to flow regulation may vary considerably depending upon the purpose and manner of operation of the dam. Construction of a dam has a direct impact on the downstream flow and sediment regime. Channel adjustments to the altered flow duration and sediment loads include changes in the bed material (armoring), bed elevation, channel width, planform, and vegetation. Lane's balance (Figure 3.19) indicates that a reduction in the discharge and sediment load, as might be expected downstream of a dam, tends to produce counter-acting results. Consequently, the response of a channel system to dam construction is extremely complex. The specific channel response will depend upon the magnitude of changes in the flow duration and sediment loads, and the existing channel regime downstream of the dam. Therefore, channel response downstream of a dam is very complex and may vary from stream to stream. Generally, the initial response downstream of a dam is degradation of the channel bed close to the dam and sedimentation further downstream due to increased supply from the degrading reach. This is the typical response most commonly anticipated downstream of a dam. Degradation may migrate downstream with time, but generally it is most significant during the first few years following closure of the dam. In some situations, a channel may shift from a degradational to an aggradational phase in response to slope flattening due to degradation, increased sediment inputs from tributaries and bed and bank erosion, and reduction in the dominant discharge.

System instability can also be introduced by the diversion of water into or out of the stream. Channel diversion structures are designed to divert a portion of the water and/or sediment from a

stream and deliver it to another location. Diversions are often needed for water supply, irrigation, hydropower, flood control, or environmental reasons. The system effects and complexities are similar to those downstream of major dams. According to Lane's balance the sediment load in the receiving stream will be increased due to extra, transport capacity of the increased discharge. In time, the erosion of bed sediments decreases as the slope is reduced through bed degradation.

An increase in discharge due to a flow diversion can have a significant impact on the channel plan form as well as the vertical stability. Schumm (1977) proposed a qualitative relation similar to Lane's that included meander wavelength. His relation states that:

$$Q = \frac{b\ d\ L}{S}$$

where Q is the discharge, b is the width, d is the depth, S is the slope, and L is the meander wavelength. The above relation indicates that an increase in discharge may result in an increase in the meander wavelength which would be accomplished through accelerated erosion of the streambanks. Therefore, whenever diversions such as this are proposed the potential for increased meander activity must be considered. If a stream is in the process of increasing meander wavelength, then stabilization of the bends along the existing alignment is likely to be unsuccessful and is not recommended.

Basin Wide Factors. Sometimes the changes in the controlling variables can not be attributed to a specific upstream or downstream factor, but rather are occurring on a basin-wide basis. This often results from a major land use change or urbanization. These changes can significantly modify the incoming discharge and sediment loads to a channel system. For example, urbanization can increase peak flows and reduce sediment delivery, both of which would tend to cause channel degradation in the channel system. A land use change from forest to row crop on the other hand might cause a significant increase in the sediment loading resulting in aggradation of the channel system. Unfortunately, it is difficult, if not impossible, to predict when basin wide changes such as these will occur. Therefore, the best the designer can do in most cases is to simply try to design the bank protection measures to accommodate the most likely future changes in the watershed. For instance, if there is a possibility of future urbanization in the upper watershed, then additional launching stone may be needed to protect the bank from the destabilizing impact of any future bed lowering.

3.4.2.2 Complexities and Multiple Factors

Lane's balance and other geomorphic analyses of initial morphological response to system disturbance provide a simple qualitative method for predicting the channel response to an altered condition. However, it does not take into account the magnitude of the change and the existing morphologic condition of the stream. For instance, according to Lane's balance a channel cutoff should induce degradation. While this is often the case, there are many examples where there may be no observable change in the channel morphology following the construction of cutoffs. Brice (1981) documented the stability of streams at 103 sites in different regions of the United States where channels had been relocated. He found that following

the cutoffs 52% of the channels showed no change, 32% showed improvement, and 16% exhibited channel degradation. This study indicates that predicting the channel response to cutoffs is not nearly as simple as might be inferred from Lane's balance. Therefore, the designer should always be aware of the considerable uncertainties that exist when attempting to predict, even in qualitative terms, the behavior of river systems.

Previous discussions have focused primarily on the initial response of a channel to various alterations in the watershed. However, it must be remembered that the entire watershed is connected and that changes in one location can, and often do, affect the channel stability at other locations, which in turn provides a feedback mechanism whereby the original channel response may be altered. For example, the initial response to a base level lowering due to channelization may be channel degradation. However, as this degradation migrates upstream the sediment supply to the downstream reach may be significantly increased due to the upstream bed and bank erosion. This increased sediment load coupled with the slope flattening due to the past degradation may convert the channel from a degradational to an aggradational phase. Multiple response to a single alteration has been referred to as **complex response** by Schumm (1977).

Another complicating factor in assessing the cause and effect of system instability is that very rarely is the instability a result of a single factor. In a watershed where numerous alterations (dams, levees, channelization, land use changes, etc.) have occurred, the channel morphology will reflect the integration of all these factors. Unfortunately, it is extremely difficult and often impossible to sort out the precise contributions of each of these components to the system instability. The interaction of these individual factors coupled with the potential for complex response makes assessing the channel stability and recommending channel improvement features, such as bank protection, extremely difficult. There are numerous qualitative and quantitative procedures that are available. Regardless of the procedure used, the designer should always recognize the limitations of the procedure, and the inherent uncertainties with respect to predicting the behavior of complex river systems.

3.4.3 LOCAL INSTABILITY

For this discussion local instability refers to bank erosion that is not symptomatic of a dis-equilibrium condition in the watershed (i.e., system instability) but results from site-specific factors and processes. Perhaps the most common form of local instability is bank erosion along the concave bank in a meander bend which is occurring as part of the natural meander process. Local instability does not imply that bank erosion in a channel system is occurring at only one location or that the consequences of this erosion are minimal. As discussed earlier, erosion can occur along the banks of a river in dynamic equilibrium. In these instances the local erosion problems are amenable to local protection works such as bank stabilization measures. However, local instability can also exist in channels where severe system instability exists. In these situations the local erosion problems will probably be accelerated due to the system instability, and a more comprehensive treatment plan will be necessary.

3.4.3.1 Overview of Meander Bend Erosion

Depending upon the academic training of the individual, streambank erosion may be considered as either a hydraulic or a geotechnical process. However, in most instances the bank retreat is the result of the combination of both hydraulic and geotechnical processes. The material may be removed grain by grain if the banks are non-cohesive (sands and gravels), or in aggregates (large clumps) if the banks are composed of more cohesive material (silts and clays). This erosion of the bed and bank material increases the height and angle of the streambank which increases the susceptibility of the banks to mass failure under gravity. Once mass failure occurs, the bank material will come to rest along the bank toe. The failed bank material may be in the form of a completely disaggregated slough deposit or as an almost intact block, depending upon the type of bank material, the degree of root binding, and the type of failure (Thorne, 1982). If the failed material is not removed by subsequent flows, then it may increase the stability of the bank by forming a buttress at the bank toe. This may be thought of as a natural form of toe protection, particularly if vegetation becomes established. However, if this material is removed by the flow, then the stability of the banks will be again reduced and the failure process may be repeated.

As noted above, erosion in meander bends is probably the most common process responsible for local bank retreat and, consequently, is the most frequent reason for initiating a bank stabilization program. A key element in stabilization of an eroding meander bend is an understanding of the location and severity of erosion in the bend, both of which will vary with stage and plan form geometry.

As streamflow moves through a bend, the velocity (and tractive force) along the outer bank increases. In some cases, the tractive force may be twice that in a straight reach just upstream or downstream of the bend. Consequently, erosion in bends is generally much greater than in straighter reaches. The tractive force is also greater in tight bends than in longer radius bends. This was confirmed by Nanson and Hickin (1986) who studied the migration rates in a variety of streams, and found that the erosion rate of meanders increases as the radius of curvature to width ratio (r/w) decreased below a value of about 6, and reached a maximum in the r/w range of 2 to 3. Biedenharn *et al*. (1989) studied the effects of r/w and bank material on the erosion rates of 160 bends along the Red River in Louisiana and also found that the maximum erosion rates were observed in the r/w range of 2 to 3. However, the considerable scatter in their data indicate that other factors, particularly bank material composition, were also modifying the meander process.

The severity and location of bank erosion also changes with stage. At low flows, the main thread of current tends to follow the concave bank alignment. However, as flow increases, the flow tends to cut across the convex bar to be concentrated against the concave bank below the apex of the bend. Friedkin (1945) documented this process in a series of laboratory tests on meandering in alluvial rivers. Because of this process, meanders tend to move in the downvalley direction, and the zone of maximum erosion is usually in the downstream portion of the bend due to the flow impingement at the higher flows. This explains why the protection of the downstream portion of the bend is so important in any bank stabilization scheme. The material eroded from the outer bank is transported downstream and is generally deposited in the next crossing or point bar. This process also results in the deposition of sediment along the upper portion of the

concave bank. This depositional feature is often a good indicator of the upstream location to start a bank protection measure.

3.4.3.2 Streambank Erosion and Failure Processes

The terms streambank erosion and streambank failure are often used to describe the removal of bank material. **Erosion** generally refers to the hydraulic process where individual soil particles at the bank's surface are carried away by the **tractive force** of the flowing water. The tractive force increases as the water velocity and depth of flow increase. Therefore, the erosive forces are generally greater at higher flows. **Streambank failure** differs from erosion in that a relatively large section of bank fails and slides into the channel. Streambank failure is often considered to be a geotechnical process. A detailed discussion of the erosion and failure processes discussed below is provided by Thorne (1993).

Identifying the processes responsible for bank erosion is not an easy task and often requires some training. The primary erosion processes are parallel flow, impinging flow, piping, freeze/thaw, sheet erosion, rilling/gullying, wind waves, and vessel forces. These erosional forces are illustrated in Figures 3.24 through 3.30 and discussed below.

Parallel flow erosion is the detachment and removal of intact grains or aggregates of grains from the bank face by flow along the bank. Evidence includes: observation of high flow velocities close to the bank; near-bank scouring of the bed; under-cutting of the toe/lower bank relative to the bank top; a fresh, ragged appearance to the bank face; absence of surficial bank vegetation.

Impinging flow erosion is detachment and removal of grains or aggregates of grains by flow attacking the bank at a steep angle to the long-stream direction. Impinging flow occurs in braided channels where braid-bars direct the flow strongly against the bank, in tight meander bends where the radius of curvature of the outer bank is less than that of the channel centerline, and at other locations where an in-stream obstruction deflects and disrupts the orderly flow of water. Evidence includes: observation of high flow velocities approaching the bank at an acute angle to the bank; braid or other bars directing the flow towards the bank; tight meander bends; strong eddying adjacent to the bank; near-bank scouring of the bed; under-cutting of the toe/lower bank relative to the bank top; a fresh, ragged appearance to the bank face; absence of surficial bank vegetation.

Piping is caused by groundwater seeping out of the bank face. Grains are detached and entrained by the seepage flow (also termed sapping) and may be transported away from the bank face by surface run-off generated by the seepage, if there is sufficient volume of flow. Piping is especially likely in high banks or banks backed by the valley side, a terrace, or some other high ground. In these locations the high head of water can cause large seepage pressures to occur. Evidence includes: pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones and layers; run-out deposits of eroded material on the lower bank. Note that the effects of piping erosion can easily be mistaken for those of wave and vessel force erosion (Hagerty, 1991a,b).

Figure 3.24 Erosion Generated by Parallel Flow

Figure 3.25 Erosion Generated by Impinging Flow

Figure 3.26 Erosion Generated by Piping

Figure 3.27 Erosion Generated by Freeze/Thaw

Figure 3.28 Sheet Erosion with Rilling and Gullying

Figure 3.29 Erosion Generated by Wind Waves

Figure 3.30 Erosion Generated by Vessel Forces

Freeze/thaw is caused by sub-zero temperatures which promote freezing of the bank material. Ice wedging cleaves apart blocks of soil. Needle-ice formation loosens and detaches grains and crumbs at the bank face. Freeze/thaw activity seriously weakens the bank and increases its erodibility. Evidence includes: periods of below freezing temperatures in the river valley; a loose, crumbling surface layer of soil on the bank; loosened crumbs accumulated at the foot of the bank after a frost event; jumbled blocks of loosened bank material.

Sheet erosion is the removal of a surface layer of soil by non-channelized surface run-off. It results from surface water draining over the bank edge, especially where the riparian and bank vegetation has been destroyed by encroachment of human activities. Evidence includes: surface water drainage down the bank; lack of vegetation cover, fresh appearance to the soil surface; eroded debris accumulated on the lower bank/toe area.

Rilling and gullying occurs when there is sufficient uncontrolled surface run-off over the bank to initialize channelized erosion. This is especially likely where flood plain drainage has been concentrated (often unintentionally) by human activity. Typical locations might be near buildings and parking lots, stock access points and along stream-side paths. Evidence includes: a corrugated appearance to the bank surface due to closely spaced rills; larger gullied channels incised into the bank face; headward erosion of small tributary gullies into the flood plain surface; and eroded material accumulated on the lower bank/toe in the form of alluvial cones and fans.

Wind waves cause velocity and shear stresses to increase and generate rapid water level fluctuations at the bank. They cause measurable erosion only on large rivers with long fetches which allow the build up of significant waves. Evidence includes: a large channel width or a long, straight channel with an acute angle between eroding bank and longstream direction; a wave-cut notch just above normal low water plane; a wave-cut platform or run-up beach around normal low-water plane. Note that it is easy to mistake the notch and platform produced by piping and sapping for one cut by wave action (Hagerty, 1991a,b).

Vessel forces can generate bank erosion in a number of ways. The most obvious way is through the generation of surface waves at the bow and stern which run up against the bank in a similar fashion to wind waves. In the case of large vessels and/or high speeds these waves may be very damaging. If the size of the vessel is large compared to the dimensions of the channel, hydrodynamic effects produce surges and drawdown in the flow. These rapid changes in water level can loosen and erode material on the banks through generating rapid pore water pressure fluctuations. If the vessels are relatively close to the bank, propeller wash can erode material and re-suspend sediments on the bank below the water surface. Finally, mooring vessels along the bank may involve mechanical damage by the hull. Evidence includes: use of river for navigation; large vessels moving close to the bank; high speeds and observation of significant vessel-induced waves and surges; a wave-cut notch just above the normal low-water plane; a wave-cut platform or "spending" beach around normal low-water plane. Note that it is easy to mistake the notch and platform produced by piping and sapping for one cut by vessel forces (Hagerty, 1991a,b).

Ice rafting erodes the banks through mechanical damage to the banks due to the impact of ice-masses floating in the river and due to surcharging by ice cantilevers during spring thaw. Evidence includes: severe winters with river prone to icing over; gouges and disruption to the bank line; toppling and cantilever failures of bank-attached ice masses during spring break-up.

Other erosion processes (trampling by stock, damage by fishermen, etc.) could be significant but it is impossible to list them all.

Serious bank retreat often involves geotechnical bank failures as well as direct erosion by the flow. Such failures are often referred to as "bank sloughing" or "caving," but these terms are poorly defined and their use is to be discouraged. Examples of different modes of geotechnical stream bank failure include soil fall, rotational slip, slab failure, cantilever failure, pop-out failure, piping, dry granular flow, wet earth flow, and other failure modes such as cattle trampling (Figures 3.31 through 3.39). Each of these is discussed below.

Soil/rock fall occurs only on a steep bank where grains, grain assemblages or blocks fall into the channel. Such failures are found on steep, eroding banks of low operational cohesion. Soil and rock falls often occur when a stream undercuts the toe of a sand, gravel or deeply weathered rock bank. Evidence includes: very steep banks; debris falling into the channel; failure masses broken into small blocks; no rotation or sliding failures.

Shallow slide is a shallow seated failure along a plane somewhat parallel to the ground surface. Such failures are common on banks of low cohesion. Shallow slides often occur as

Figure 3.31 Soil Fall

Figure 3.32 Rotational Slip

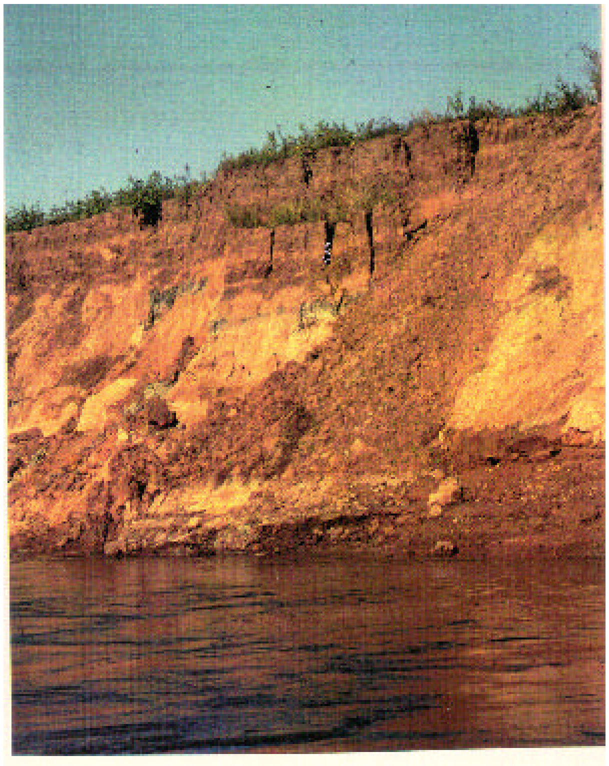


Figure 3.33 Slab Failure

Figure 3.34 Cantilever Failure

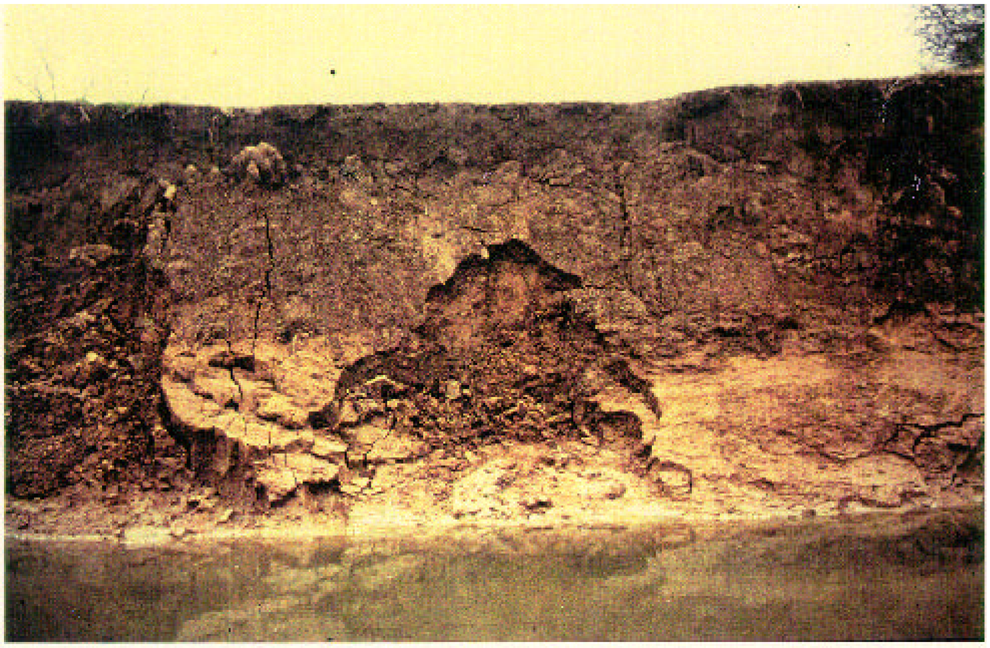


Figure 3.35 Pop-out Failure

Figure 3.36 Piping

Figure 3.37 Dry Granular Flow

Figure 3.38 Wet Earth Flow

Figure 3.39 Cattle Trampling

secondary failures following rotational slips and/or slab failures. Evidence includes: weakly cohesive bank materials; thin slide layers relative to their area; planar failure surface; no rotation or toppling of failure mass.

Rotational slip is the most widely recognized type of mass failure mode. A deep seated failure along a curved surface results in back-tilting of the failed mass toward the bank. Such failures are common in high, strongly cohesive banks with slope angles below about 60º. Evidence includes: banks formed in cohesive soils; high, but not especially steep, banks; deep seated, curved failure scars; back-tilting of the top of failure blocks towards intact bank; arcuate shape to intact bank line behind failure mass.

Slab-type block failure is sliding and forward toppling of a deep seated mass into the channel. Often there are deep tension cracks in the bank behind the failure block. Slab failures occur in cohesive banks with steep bank angles greater than about 60º. Such banks are often the result of toe scour and under-cutting of the bank by parallel and impinging flow erosion. Evidence includes: cohesive bank materials; steep bank angles; deep seated failure surface with a planar lower slope and nearly vertical upper slope; deep tension cracks behind the bank-line; forward tilting of failure mass into channel; planar shape to intact bank-line behind failure mass.

Cantilever failure is the collapse of an overhanging block into the channel. Such failures occur in composite and layered banks where a strongly cohesive layer is underlain by a less resistant one. Under-mining by flow erosion, piping, wave action and/or pop-out failure leaves an overhang which collapses by a beam, shear or tensile failure. Often the upper layer is held together by plant roots.

Evidence includes: composite or layered bank stratigraphy; cohesive layer underlain by less resistant layer; under-mining; overhanging bank blocks; failed blocks on the lower bank and at the bank toe.

Pop-out failure results from saturation and strong seepage in the lower half of a steep, cohesive bank. A slab of material in the lower half of the steep bank face falls out, leaving an alcove-shaped cavity. The over-hanging roof of the alcove subsequently collapses as a cantilever failure. Evidence includes: cohesive bank materials; steep bank face with seepage area low in the bank; alcove shaped cavities in bank face.

Piping failure is the collapse of part of the bank due to high groundwater seepage pressures and rates of flow. Such failures are an extension of the piping erosion process described previously, to the point that there is complete loss of strength in the seepage layer. Sections of bank disintegrate and are entrained by the seepage flow (sapping). They may be transported away from the bank face by surface run-off generated by the seepage, if there is sufficient volume of flow. Evidence includes: pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones; run-out deposits of eroded material on the lower bank or beach. Note that the effects of piping failure can easily be mistaken for those of wave and vessel force erosion.

Dry granular flow describes the flow-type failure of a dry, granular bank material. Other terms for the same mode of failure are ravelling and soil avalanche. Such failures occur when a noncohesive bank at close to the angle of repose is undercut, increasing the local bank angle above the friction angle. A carpet of grains rolls, slides and bounces down the bank in a layer up to a few grains thick. Evidence includes: noncohesive bank materials; bank angle close to the angle of repose; undercutting; toe accumulation of loose grains in cones and fans.

Wet earth flow failure is the loss of strength of a section of bank due to saturation. Such failures occur when water-logging of the bank increases its weight and decreases its strength to the point that the soil flows as a highly viscous liquid. This may occur following heavy and prolonged precipitation, snow-melt or rapid drawdown in the channel. Evidence includes: sections of bank which have failed at very low angles; areas of formerly flowing soil that have been preserved when the soil dried out; basal accumulations of soil showing delta-like patterns and structures.

Other failure modes could be significant, but it is impossible to list them all. Cattle trampling is just one example of a common failure mode.

3.5 CLOSING

In planning a project along a river or stream, awareness of even the fundamentals of geomorphology and channel processes allows you to begin to see the relationship between form and process in the landscape. Go into the field and take notes, sketches, pictures - and above all, observe carefully, think about what you are seeing, and use this information to infer the morphological status of the river. When you are in the field, look at your surroundings and try to establish a connection between what

you see (form) and why it is there (process). Then you will begin to have some understanding and can perhaps begin to predict what sort of changes may result if your project alters the flow patterns. Then you are beginning to think like a geomorphologist. Dr. Einstein (1972) said in the closing comments of his retirement symposium:

> *It is in the field where we can find out whether our ideas are applicable, where we can find out what the various conditions are that we have to deal with, and where we can also find out what the desired improvements are.*

CHAPTER 4

CHANNELIZATION AND CHANNEL MODIFICATION ACTIVITIES AND IMPACTS

This chapter introduces general categories of channelization and channel modification projects along with activities implemented to achieve project goals and to offset adverse impacts.

Channel modification activities have a variety of impacts on riverine processes and the associated riparian ecology and terrestrial environment. These activities can impact river morphology and related ecology for many years after construction. Projects undertaken to straighten, enlarge, or relocate the channel in alluvial river systems can initiate channel instability that ultimately leads to stream bank instability. The resulting bed and bank erosion produces changes in the rates and paths of sediment erosion, transport, and deposition within the river system. Accelerated erosion and sedimentation processes resulting from channel modification activities can be detrimental to the infrastructure such as bridges or roadways. Bank erosion and bank failure results in a loss of riparian habitats as well as commercially valued real estate adjacent to the stream. Degradation of the bed results in a loss of native substrate and a reduction in the diversity of aquatic habitats. The downstream migration and subsequent deposition of sediments resulting from channel and streambank erosion can adversely impact the in-stream habitat of flora and fauna. Shields and Palermo (1982) present the following six areas of adverse environmental effects of channelization:

1) Loss of aquatic habitat or reduction in aquatic habitat diversity;
2) Loss of terrestrial habitat or reduction in terrestrial habitat diversity;
3) Increased sediment concentrations and turbidity due to bed and bank instability;
4) Reduction of aesthetic value of streams and riparian habitat;
5) Water quality degradation, principally due to increasing water temperature and suspended sediment concentration; and
6) Changes in the stream related hydrology such as fluctuating water levels, draining of wetlands, and increasing uniformity of flow conditions.

Channel modification activities have deprived wetland and estuarine shorelines of enriching sediments, changed the ability of natural systems to both absorb hydraulic energy and filter pollutants from the surface waters, and caused interruptions in the different life stages of aquatic organisms (Sherwood *et al.*, 1990). A frequent result of channelization and channel modification activities is a diminished suitability

of in-stream and riparian habitat for fish and wildlife. Clearing of banks along waterways has eliminated in-stream and riparian habitats, decreased the quantity of organic matter entering aquatic systems, altered the water temperature, and increased the movement of non-point source pollutants from the upper reaches of watersheds into river systems and ultimately coastal waters. Excavation projects can result in reduced flushing, increased suspended sediment loads, lowered dissolved oxygen levels, saltwater intrusion, loss of riparian vegetation, accelerated discharge of pollutants, and changed physical and chemical characteristics of bottom sediments in surface waters surrounding channelization or channel modification projects. Reduced flushing, in particular, can increase the deposition of fine-grained sediments and associated organic materials or other pollutants. Confining river channels to reduce overbank flooding results in a reduction of sediment needed to nourish riverine and estuaries, wetlands and riparian areas and accelerates the delivery of suspended sediments to coastal and near coastal waters during high flow events. Construction activities that support channel modification projects can have adverse impacts on both river processes and the environment. Clearing of terrestrial and riparian vegetation results in a loss of habitat and can accelerate runoff and subsequent erosion of the banks.

Channel modification projects are designed and implemented to provide a benefit to the surrounding areas, whether for preventive measures such as flood control or economic measures such as mining. The adverse impacts associated with channel modification activities can be systematically addressed with specific remedial practices to reduce or eliminate the severity of impacts.

4.1 CHANNELIZATION AND CHANNEL MODIFICATION PROJECT CATEGORIES

Channel modification projects will involve activities that alter channel parameters such as length, width, depth, slope, discharge, sediment size, or sediment discharge. In Chapter 3, the concepts of channel stability and system equilibrium were discussed. Lane's Balance was presented as a methodology for qualitatively determining channel stability as discharge, sediment discharge, slope, and sediment size are changed due to channel modification activities. The following project category descriptions provide a broad overview of the need for projects, and the general activities that are implemented to accomplish the project goals. The concepts embedded in Lane's Balance apply to channel modification activities that involve changing the variables critical for channel stability.

4.1.1 FLOOD CONTROL AND DRAINAGE

Channel modification activities for flood control are designed and implemented to insure that flood flows remain within acceptable levels. Methods are implemented to either convey, confine, or control flood discharge. The projects reduce the channel resistance to flow, provide flood water storage, bypass the flood flows, or artificially confine the flows within the original channel. This can generally be accomplished by removing obstructions from the channel, straightening the channel, enlarging or deepening the channel, or constructing impoundments, diversion channels, or levee systems. Although these are logical methodologies to consider, and will improve flood defense, each can potentially alter the equilibrium of the channel.

4.1.2 NAVIGATION

The primary objective of channel modification to support navigation is to establish and maintain an adequate navigable depth. Rivers that have sufficient natural flows to support commercial navigation utilize dredging operations and in-channel training structures to remove accumulated sediment within the channel to maintain a navigable depth. River systems with flows regulated by Dams provide opportunities for navigation by providing periodic adequate flows by regulated releases through the dam. It is not only critical to maintain project depth, but also control migration of the channel thalwag, bendways, and channel sinuosity to insure project dimensions for safe navigation.

4.1.3 SEDIMENT CONTROL

Although sediment transport in an alluvial channel is considered a natural phenomenon, sediment can be classified as a non-point pollutant, and sediment may be present in excessive quantities that have damaging effects on the environmental and physical aspects of river systems. Excessive sedimentation can result from erosion and runoff resulting from man-induced practices such as mining, farming, development, construction, and channel maintenance activities. Excessive sediment can result from degradation of the channel bed and streambank erosion or failure resulting from river system instabilities. In navigable waterways, millions of dollars are expended annually to remove accumulated sediments from the navigation channel. Deposition of sediments in sensitive biological communities can result in a reduction of substrate diversity that in turn can affect the population of benthic invertebrates. Conversely, a reduction of sediment supplied to wetland environments adjacent to river systems can result in land loss. Kesel (1988) reports that the decrease in suspended sediment loads in the Mississippi River combined with the construction of artificial levees has resulted in an accelerating decline in Louisiana coastal land area. Excessive sedimentation reduces the capacity of flood control channels and can impact the infrastructure. Potable water supply operations can be severely impacted by excessive suspended sediment concentrations. Sediment control activities include sediment removal (dredging), the implementation of streambank and channel stability projects, better construction methods, trapping or storing sediments, structures for diverting flow, construction of sediment retention dams, and increased use of protective vegetation.

4.1.4 INFRASTRUCTURE PROTECTION

Frequently, man-made structures such as buildings, bridges, and control structures are located adjacent to channels or tributaries. Channelization and channel modification projects can accelerate erosion processes and lead to damage or complete failure of an adjacent infrastructure. Excessive sediment transport and sedimentation can impact water supply and diversion operations. Local channel and streambank stabilization activities are implemented to halt channel and streambank degradation, and subsequently protect structural foundations. Typical channel and bank protection activities include armoring techniques such as lining banks and channels with riprap, the use of grade control structures to stabilize eroding channels, and the use of training structures such as dikes to divert flows that impinge on structural

foundations, Channel realignment activities may be required to isolate the structure from degradational areas.

4.1.5 MINING

Mining operations typically associated with river systems include sand, gravel, phosphate, metals and other materials. Mining operations can affect in-channel, riparian, and terrestrial processes. Removal of large quantities of sand, gravel, or ore from the channel can lower the bed, thus initiate a bed degradational process that proceeds upstream. The increased sediment load is transported and deposited downstream. Additionally, mining operations can significantly increase suspended sediment loads through the mining process, and through the destruction of protective stream side and terrestrial vegetation. Waste products of mining operations may be deposited adjacent to the streams, and may be re-deposited in the channel during high water or storm events. Generally, both hydraulic and mechanical dredging activities are associated with riverine mining activities.

4.1.6 CHANNEL AND BANK INSTABILITY

Unstable riverine systems result in accelerated loss of stream side habitats due to bed and bank erosion, adverse impacts on aquatic habitats from increased sedimentation and turbidity, and a general decreases in the recreational value of the system. Channel modification activities are implemented to halt or slow down processes associated with instability such as bank erosion and channel degradation. Grade control structures are constructed in channels to stop the degradational process from proceeding upstream. Grade control structures include simple bed control structures, concrete drop structures, channel linings or drop pipes (Watson *et al*., 1998). Stream banks are stabilized to halt erosion and bank failures resulting from localized effects or overall system instability. Typical channel and bank stability techniques include the use of surface armor for erosion protection, indirect methods such as dikes, weirs, and retards for redirecting flows away from affected areas, and the use of vegetation for either direct or indirect erosion protection.

4.1.7 HABITAT IMPROVEMENT AND ENHANCEMENT

Associated with channel modification projects are activities to improve an in-stream and riparian habitat. Existing channels that have been modified for purposes such as flood control and drainage frequently experience a loss of stream side and terrestrial vegetation, bed substrate, and in-stream habitat diversity. Loss of stream side vegetation can result in an increase in water temperature due to a reduction of shade, which impacts aquatic species that can only tolerate a narrow range of temperatures. Additionally, loss of protective vegetation increases stream bank erosion and transport of sediment into the stream. Increased sedimentation or erosion rates resulting from channel instability can replace the existing substrate with a more uniform substrate that is not conducive to a diverse colonization of aquatic invertebrates. Habitat improvement activities include re-introducing vegetation along the stream bank to

provide needed protection and placing artificial structures such as boulders, gravel, or sills into the channel to provide the needed channel bed diversity for aquatic organisms to thrive.

4.1.8 RECREATION

National policy requires full consideration of recreation as a project purpose during the planning of water resources development projects (Office, Chief of Engineers (OCE), 1982). Incorporation of recreational features in a channel modification plan not only provides project benefits that offset any adverse impacts, but also generates positive public perception of the project. Many recreational facilities on channel modification projects are cost shared with local sponsors. Recreational activities associated with channel modification projects include creation of lakes or reservoirs for water sports such as boating and fishing, nature trails or study areas, and campgrounds. Channel improvements may be initiated to improve fisheries and recreational boating. Channels should be designed to provide adequate access, suitable low flow depths, and as few obstructions as possible for recreational navigation (Nunnally and Shields, 1985).

4.1.9 FLOW CONTROL FOR WATER SUPPLY

Channel modification activities associated with water supply include the creation of impoundments and diversion canals with associated flow control structures. Dams and reservoirs have multiple uses such as municipal water supply, flood water storage, sediment storage, and recreation. Diversion canals supply water for irrigation, municipal water supply, and divert flood waters.

The construction of dams and associated reservoirs interrupts the natural sediment discharge of the pre-dam river system. Dams alter the flow and sediment regime that can result in significant morphological changes in downstream reaches. The bulk of incoming sediment is stored in the reservoir. Thus, the sediment discharge downstream is minimal. Additionally, a reduction in discharge can occur during dam operations to reduce flood peaks downstream. In accordance with Lane's Balance (Chapter 3), assuming that the flow and sediment size remains constant, the reduction in sediment discharge results in a decreasing channel slope. The reduction of water discharge, however, will allow a steeper slope to exist. Therefore, predicting the response of a downstream channel to dam flow control is extremely difficult due to these complexities (USACE, GDM-54, 1990a). Additionally, water discharged through the dam typically is at a lower temperature, thus possibly limiting the habitat acceptable for native aquatic species.

Diversion canals can also impact the stability of the river system by diverting water and sediment from the main channel. Impacts include degradation of the bed and associated bank erosion and failure and a reduction in habitat diversity. Morphological impacts occur for both the diversion channel and the source stream. A reduction in flow in the main stream due to the diversion results in a decreased sediment discharge, thus deposition occurs downstream of the diversion, assuming that the diversion does not change the bed material load in the main channel. If a substantial bed material load is diverted without a

proportionate reduction in flow, then the bed and banks of the main stream may erode and enlarge the crossection.

4.2 CHANNEL MODIFICATION ACTIVITIES AND ASSOCIATED IMPACTS

The previous section described the general types of channel modification projects and related impacts. This section will address specific activities that are implemented to achieve project goals or address project environmental concerns. As previously described, activities that alter channel geometry may create stability problems. Each of the following activities is implemented to achieve some level of benefit to the surrounding community or region. Both the benefits and adverse impacts for each activity are described.

4.2.1 SNAGGING AND CLEARING

Snagging and clearing activities are implemented to increase discharge capacity of channels for flood control and drainage purposes and to prevent hazards to navigation or bridges. The increased flow resistance due to the presence of vegetation and debris may increase the frequency and duration of overbank flows. The goal of the practice is to remove sufficient vegetation, debris, logs, sediment blockages, large rocks, and other obstructions from the channel and adjacent banks to decrease flow resistance. These obstructions retard flow by reducing the effective cross-sectional area of the channel, increasing the channel roughness, and trapping additional debris, particularly during high flows (Shields and Palermo, 1982). Various methods are used for removing channel debris and obstructions.

For flood control on small streams, conventional practice has been to remove all obstructions from the channel and to clear all significant vegetation within a specified width on both sides of the channel (Nunnally and Shields, 1985). For small streams, clearing of the channel is accomplished with heavy equipment such as bulldozers. On navigable streams, a floating plant may be utilized for the clearing operation. Comprehensive guidelines and practices for removing obstructions from streams are presented by the Stream Renovation Guidelines Committee, The Wildlife Society and American Fisheries Society (1983). This guidance is intended to aid in correcting stream flow problems caused by obstructions in an environmentally sound manner and to maintain natural stream characteristics.

4.2.1.1 Hydraulic Effects

The extent of the effect of clearing and snagging operations on channel discharge capacity is related to the degree of blockage prior to clearing. Potential stability and sedimentation responses to clearing and snagging are associated mainly with increased velocities, increased transport capacity, and with removal of vegetation that may have acted locally as erosion protection. Effects on stability may be adverse in some locations and beneficial in others. The qualitative effect on stability was demonstrated using Lanes Balance as described in Chapter 3.

4.2.1.2 Environmental Effects

The removal of snags and debris reduces habitat diversity in the channel. Increased velocities allow deposits of leaves, twigs, and fine grained sediments to be washed downstream. These deposits are an important habitat for many benthic species and in channels with sandy, shifting substrates form the only suitable habitat. Removal of the vegetative canopy from streambanks may result in decreased shade and resultant high stream temperatures, decreased input of organic matter such as leaves, and increased photosynthesis in the stream (Shields and Palermo, 1982). The removal of snags increases the mean velocity of the stream, which may affect plankton production or erode away fine sediment that provides substrates for specific kinds of benthic organisms. Impacts on the macro invertebrate community will ultimately affect fish populations that depend on invertebrates for food. The change in food resources may result in a fish population reduction or an undesirable change in species composition. Additionally, fish may be adversely affected by the removal of snags that serve as cover and shelter.

Clearing large amounts of terrestrial vegetation can affect terrestrial communities. Populations of mammals and reptiles that utilize streambank vegetation for shelter and feeding areas will decrease accordingly. Studies in Vermont (Dodge *et al*., 1977) and Mississippi (Arner *et al*., 1976) found mammal track counts along natural streams were almost twice as great as mammal track counts along streams that had vegetation removed by snagging and clearing operations.

4.2.1.3 Remedial Practices

Adverse environmental effects may be greatly reduced with little loss in flood control by limiting the type and amount of snags and vegetation removed and by using construction methods that create only minimal disturbance (Nunnally and Shields, 1985). Specific obstructions are designated for removal while environmentally valuable logs, snags, and vegetation that have little or no effects on flow capacity are left in place. Planning and design of clearing and snagging operations should include an evaluation of the importance of the canopy to the stream community. Specifications may be written to restrict the amount and type of terrestrial vegetation to be removed. Additionally, the type of equipment used and the access to the stream can be controlled by specification.

4.2.1.4 Operation and Maintenance of Snagging and Clearing Projects

Clearing and snagging operations require more frequent inspections than other flood control projects, particularly in regions having long growing seasons. Regrowth of vegetation on cleared channel sides and top banks can significantly increase resistance more than one or two years growing seasons. Clearing and snagging projects should never be implemented on unstable streams with wooded banks because of the high probability of future bank failures with the subsequent re-introduction of debris into the stream.

4.2.2 CHANNEL ENLARGEMENT

Channel enlargement activities are generally implemented when a larger increase in channel flow capacity is required. Snagging and clearing operations are undertaken when decreasing flow resistance can achieve the desired effect on flow capacity. Channel clean out involves changing the channel width, depth, or both to support both flood control and navigation efforts. In small non-navigable streams, the channel is generally accessed from the bank, with dragline operations used to increase channel width and depth. For navigable streams or rivers, a floating dredge plant, either hydraulic or mechanical, is used. The degree of excavation can range from removal of a few shoals to an order of magnitude change in channel geometry. The design of the new channel geometry is based on the desired flow rates, sediment transport characteristics, and bank stability.

4.2.2.1 Hydraulic Effects

Channel enlargement operations result in a significant change in flow capacity, and potentially impact channel stability. These channel modifications typically increase the cross sectional area (channel width and depth) and decrease channel roughness due to removal of debris and vegetation, thus increasing flow capacity. The concept of Lanes Balance presented earlier in chapter 3 indicates that for equilibrium the supply of sediment must equal the flow capacity. For high flow events, the probable result of increasing flowrate with the same sized sediment in the channel is a degradation of the bed and increased bank erosion. Both upstream and downstream reaches are affected. The increased velocity in the enlarged reach will result in scour from the bed and banks upstream, with the sediment delivered and deposited downstream. For nominal flows that characterize the majority of the flow events, widening the river results in an over-designed channel with an increased flow area. This results in reduced velocities, thus decreasing the sediment transport capacity that results in sediment deposition. In severe cases of over-widened streams, channel bars or braided flow can occur at low discharges (Brookes, 1988). Deepening the channel can lower tributary base levels, thus increasing tributary slopes. According to Lane’s Balance, if the slope is steepened, the sediment transport rate must increase for stability. This results in an upstream migration of degradation of the channel bed often referred to as headcutting. Material excavated from the channel and associated banks during cleanout operations can be used to build berms along the banks for additional flood protection, but may further confine flows, thus exacerbating stability problems.

4.2.2.2 Environmental Effects

Like snagging and clearing, removing material from the banks and the channel decreases habitat diversity, thus negatively impacting the aquatic community. Typically, an enlarged channel will have a uniform cross section, which destroys pools and riffles associated with natural channels. The associated loss of habitat diversity can manifest itself by a reduction in species diversity or composition, a reduction in size, distribution, and condition of the population, or unnatural seasonal variations in populations (Gorman and Karr, 1978). The uniform geometry along with the banks denuded of vegetation gives the appearance of a uniform, linear ditch that has very little aesthetic value. When channel clean out operations are conducted from the bank, riparian vegetation can be damaged or removed that reduces habitats and

potentially increases streambank erosion. Low flows in enlarged channels may not have the pools necessary for aquatic organisms to thrive. Because of the low velocities in enlarged channels, vegetation may invade the channel and create a future channel maintenance problem.

Material excavated from the enlargement operations may be used to construct levees as a management tool for providing additional flood protection. In Louisiana, material excavated from channels was used to prevent saltwater intrusion into a brackish coastal marsh (Scott, 1972). Levees will reduce overbank flows, thus potentially interfering with groundwater recharge and floodplain plant diversity. Shields and Palermo (1982) list the following environmental consequences that should be considered when enlarging a stream:

1) Placement of excavated or dredged material;
2) Cross-sectional shape and uniformity;
3) Changes in substrate and substrate diversity;
4) Removal of channel armor;
5) High and low flow depths and velocities in the modified channel;
6) Increased peak flows downstream; and
7) Changes in stream-floodplain-groundwater interactions.

4.2.2.3 Remedial Practices

A method of enlargement that can reduce instability problems is the use of side berm cuts to form a two-stage channel (USACE, EM 1110-2-1418, 1994). Although it has the disadvantage of using more adjacent land than simply enlarging the channel, it is more effective in conveying bed material because higher velocities are maintained at moderate discharges. The level of the berms should correspond to the channel forming discharge under modified conditions. The side berm design is described by Nunnally and Shields (1985) as a high flow channel.

Before any environmental improvement projects are undertaken, the system stability must be addressed. The key to successful project implementation is to design a stable channel before enlarging operations take place. A complete analysis of the hydrologic, hydraulic, and sediment transport requirements of the enlarged channel should be evaluated before channel modifications commence. Anticipated stability problems can then be addressed and resolved to prevent problems upstream and downstream of the affected reach. A systematic approach to channel rehabilitation is presented in Chapter 2 of this manual. Placing environmental enhancements such as artificial structures in an unstable reach of the channel can result in a total loss of the structures or inefficient or ineffective operation.

Efforts to reduce environmental impacts should be incorporated into the design of channel enlargement projects. Consideration should be given to reproducing or improving the habitat diversity of the existing stream, or preserving a part of the natural stream. In-stream diversity can be improved in post-construction channels by use of artificial structures. The purpose of artificial structures is to restore habitat and habitat diversity conducive to the growth and re-population of desirable species. In enlarged channels

with shallow depths and uniform unvarying substrates, artificial structures can reproduce the diversity of the natural channel by creating alternating pool and riffle areas. Examples of artificial structures include randomly placed boulders, small check dams, artificial riffles, bank covers, and current deflectors (Shields and Palermo, 1982). Care must be taken to avoid creating additional channel instability problems due to increased roughness or scouring when using artificial structures. Single bank construction is the preferred technique for lessening environmental impacts of channel enlargement (Nunnally, 1985). The existing channel alignment is followed with enlargement confined to one side. Vegetation on the opposite bank is left undisturbed. The disturbed bank is revegetated to reduce erosion and sedimentation in the channel.

Erosion of the stream bank can be addressed with bank protection works. Concrete lined channels have been employed, but are typically much more expensive than stone covers and further reduce the in-stream and riparian habitat.

4.2.2.4 Operation and Maintenance of Channel Enlargement Projects

As with snagging and clearing projects, channel enlargement projects require periodic inspections. Effects of channel instability due to the alteration of channel geometry may need to be addressed upstream and downstream of the project. Channel stability should be monitored for signs of aggradation, degradation, and planform changes. Re-growth of vegetation may occur during periods of low flow that may require periodic maintenance.

4.2.3 CHANNEL REALIGNMENT

Channel alignment is often performed in conjunction with clearing and snagging. It is the process of taking a sinuous channel and straightening it for the purpose of flood control, infrastructure protection, or navigation. Additionally, channel realignment activities are implemented to reduce loss of land by meander migration. Channel realignment can be implemented in varying degrees. An improved stream alignment can be accomplished by removing shoaling areas such as point bars. For flood control applications, the channel may be straightened to increase the slope and reduce flow resistance, thus increasing the capacity of the stream to convey floodwaters. This practice may involve cutting off large meanders of the river, thus actually shortening the river. The resulting cutoff generally results in slope adjustments for the affected reach. In some environments, streams with stable meanders, flat slopes and erosion resistant boundaries can withstand considerable realignment without serious impacts on system stability (Brice, 1981). In other systems, it can lead to serious problems of channel degradation, bank erosion, and tributary incision. Lane (1947) describes the response of an alluvial channel to a single cutoff. The channel upstream of the cutoff will degrade as the channel slope flattens to re-establish an equilibrium slope at a lower elevation. The reach downstream of the cutoff aggrades due to the increased sediment supply from the degrading reach. A comprehensive description of the impacts of man-made cutoffs on the Lower Mississippi River is provided by Biedenharn (1995).

The benefits for flood control are increased conveyance of floodwaters. For navigation, a straight channel reduces transit time and the need for dredging point bars adjacent to bends in the river. Channel realignment may be necessary to protect an infrastructure located near or on the stream bank.

4.2.3.1 Hydraulic Effects

Changes to a fluvial system, whether man made or natural, tend to be absorbed by the system through a series of channel adjustments (Simon and Hupp, 1987). Realignment of channels by creating cutoffs generally reduces the sinuosity and increases the slope. According to Lanes's Balance, if the slope increases and the water discharge and median grain size remains constant in the stream, the sediment transport capability of the stream increases. To approach equilibrium, the additional sediment must be obtained from either bed or bank degradation. As the bed continues to degrade, the zone of increased slope will migrate upstream. The additional sediment load transported through the realigned reach will then be deposited in lower reaches where the slope was not increased. Channel erosion migrates upstream in the form of a headcut, which is a vertical discontinuity in the streambed. The headcutting process is described in Chapter 3. Bank erosion in the steepened reaches and aggradation in the lower reaches tends to increase the width/depth ratio. This sequence is the classic response to cutoffs described by Lane (1947).

4.2.3.2 Environmental Effects

The environmental impacts of realigning channels include many of the impacts of channel enlargement and snagging and clearing. Overall, the habitat diversity is reduced in the channel as well as on the banks due to access problems with heavy equipment and clearing of vegetation. The major problems unique to channel realignment are increased channel slopes due to reduction in channel length and the reduction in habitat diversity caused by creating cutoff meanders. The increased channel slope results in an increased sediment transport capacity. The additional sediment requirement is met by degradation of the bed and stream bank. The degradational process increases sediment loads and turbidity levels that are detrimental to both benthic and in-stream aquatic organisms. Sediment deposition downstream of the unstable reach may smother benthic organisms. Unstable, shifting substrates are not conducive to maintaining macro invertebrate populations. Because of the decrease in light penetration in turbid waters, photosynthesis is reduced and plant populations are impacted. Fish populations are directly impacted by the loss of food resources.

Channel realignment activities can result in a significant loss of aquatic habitats. Cutoff meanders resulting from channel straightening activities are a significant backwater habitat. If the meanders are not maintained, these will become isolated from the main channel due to sediment deposition at the confluence with the main channel. The resulting oxbow lake will eventually fill with runoff sediment and become terrestrial habitats. If the realigned channel is maintained, new meanders will not form to replace the lost aquatic habitat. A large scale reduction in aquatic habitats will reduce the productivity of the system and may impact the diversity and population of native aquatic organisms.

4.2.3.3 Remedial Practices

The environmental impacts of channel realignment should be included in project design considerations. An estimation and evaluation of the losses of aquatic and riparian habitat should be considered if cutoffs will be formed during channel realignment. Flow should be maintained, if possible, through the old meanders to prevent them from filling with sediment. The upstream migration of channel degradation due to increased slopes resulting from shortening the channel is the most significant impact on channel stability. It must be addressed before habitat restoration practices are applied. To mitigate bed and bank erosion, grade control structures and bank stabilization techniques are implemented.

4.2.3.4 Operation and Maintenance of Channel Realignment Projects

Realigned or straightened channels should be periodically inspected for signs of instability. Grade control and bank stabilization projects incorporated into the project should be inspected and maintained to insure proper function.

4.2.4 DREDGING AND MINING

Dredging is the process by that sediments are removed from channels for the purpose of maintaining existing navigation (maintenance dredging) or deepening existing channels for deep draft navigation (new work dredging). Dredging is also utilized in bays and harbors located along rivers or at the river outlets that continuously shoal with fine sediments. Additionally, dredging operations are used for mining sand and gravel from rivers. Generally, two different types of dredging operations are used for riverine dredging. Hydraulic dredging operations consist of a floating plant that removes and transports sediments from the channel bed using large centrifugal pumps. The pump suction line extends to the channel bed where the sediment is hydraulically entrained, passed through the pump, and discharged to disposal. Disposal areas can either be within banks or located at inland confined sites. For loosely flowing coarse sediments, a plain suction head is used to entrain the sediments. For more consolidated sediments, a rotating cutterhead is employed to loosen the material and feed the suction line. In some riverine environments, hopper dredges are used. The hopper dredges are deep-draft seagoing vessels used primarily for maintenance dredging in harbors or river outlets. Hopper dredges make successive passes over the problem area, deepening progressively on each pass. The pumped material is stored in hoppers in the dredge, and when fully loaded, the dredge travels to a designated dump site in the ocean. It is only effective for dredging loose, unconsolidated material.

Mechanical dredging operations are generally conducted in shallow areas containing loose or consolidated sediments. The operation involves excavating sediment with either a barge mounted power shovel (dipper) or a clamshell bucket operation. Bucket capacities range from 1 to 12 cubic yards. The material is excavated and loaded into an adjacent barge that is towed to disposal.

4.2.4.1 Hydraulic Effects

Continuous dredging causes a river bed to degrade until the balance between sediment load supplied to the river reach and the sediment transport capacity is restored (Brookes, 1988). Deepening the river channel will lower tributary base levels, thus increasing tributary slopes. Channel instability within the tributary will result in degradation of the channel bed, increased sediment transport, and ultimately deposition of sediment within the river. Channel deepening also reduces the sediment transport capability of the river, thus deepened sections act as sediment traps and encourage sediment deposition. A study reported by Griggs and Paris (1982) described increased sediment deposition due to channel deepening. Within 10 years of completion of the U.S. Army Corps of Engineers flood channel on the San Lorenzo River at Santa Cruz in California, 350,000 cubic meters of sediment had been deposited. This reduced the carrying capacity of the river from the designated 100-year flood to a 25-30 year flood. The channel had been deepened by some 0.9 to 2.1 meters below the original bed elevation.

Mining operations that remove sand and gravel from the channel bed result in a localized lowering of the bed. This has the effect of increasing the slope upstream of the mining operation that in turn increases the sediment transport capability of the river. Bed degradation advances upstream with sediment aggradation occurring downstream. If sand and gravel mining is performed at many locations along a river, the rate of sediment removed may exceed the rate of replenishment. This can result in a significant lowering of the bed that increases the potential for undermining foundations and bridge piers during major floods. Frequent mining operations can also remove the coarser fractions of sediment that are important for armoring the bed and stabilizing the banks along the river. From Lane's balance described earlier, a reduction of sediment grain size can result in degradation as the channel flattens the slope in order to satisfy the increased transport requirements.

4.2.4.2 Environmental Effects

Dredging operations may increase turbidity at the point of dredging. Suspended sediment plumes can migrate to sensitive areas such as fish and shell fish spawning grounds. Generally, hydraulic dredging operations with plain suction intakes operating in coarse sediment environment produce very little turbidity. Cutterhead dredging operations do tend to resuspend sediments around the rotating cutterhead, particularly when working in fine sediments. The turbidity generated can be minimized by reducing the speed of the cutterhead and the swing rate of the dredge ladder (suction line). Mechanical dredges have the highest probability of re-suspending sediments**.** Sediments are resuspended by leaking buckets and through the uplift of sediments from the excavation area when the bucket or dipper is raised. Environmental dredge buckets are available that have a positive pressure seal to prevent leakage.

The major impact of dredging on biological communities is the removal and subsequent changing of the substrate. For maintenance dredging in major river systems that have a continually moving and shifting bed, this is a minor concern. For new work dredging in channels that have historically had a stable substrate, the impacts can be severe and permanent. Not only is the substrate removed, the deepening will

make the area more conducive for sedimentation, and thus periodic dredging will be required to maintain project depth. A stable substrate will no longer be available thus the diversity and suitability of the habitat will be reduced, with native aquatic organisms displaced. Turbidity generated by dredging operations can impact nearby fish and shell fish spawning grounds and inhibit plant growth.

4.2.4.3 Remedial Practices

To protect adjacent sensitive areas such as spawning grounds or vegetation, restrictions can be placed on dredge operations. Restrictions on dredge type and minimum turbidity generated can be specified in dredging contracts to insure that environmentally sensitive areas are not impacted. Physical barriers such as silt screens can be used to contain the suspended sediment plume to the immediate area surrounding the dredge. Specialty dredges designed to minimize turbidity are available.

Dredging induced channel instability is similar to that resulting from channel enlargement and realignment. Grade control structures and bank stabilization practices may be necessary to address bed and bank erosion and ultimately stabilize affected reaches.

4.2.4.4 Operation and Maintenance of Dredging and Mining Projects

The river reaches that are maintained through dredging must be periodically surveyed to insure navigable depth and width. The cost of dredging can be significant. At the mouth of the Mississippi river, dredging is conducted year round. The cost of a large hydraulic dredge can cost more than $1,500 per hour of operation. At low water or after a flood event, multiple dredges may be operating continuously to insure safe navigation. Hydraulic dredges come in a variety of sizes for a variety of applications. Six inch to eight inch diameter pipeline dredges are generally used for small waterways, canals, or lakes and reservoirs, and are limited in productivity and power. Small hydraulic dredges can generally be transported to the site by overland transportation. Costs of dredging include mobilization and de-mobilization, disposal site creationandpreparation, and general operating expenses.

4.2.5 CONSTRUCTION OF LEVEES

Levees fall into the general category of embankments. Embankments, also known as flood banks, levees, bunds or stopbanks (Brookes, 1988), are constructed to artificially increase the capacity of a channel to confine high flows that otherwise would overtop the banks and spread over the floodplain. Some of the largest river systems in the world have extensive levees. Levees extend more than 1,000 km along the Nile River and 1,400 km on the Red River in Vietnam. In the United States, levees are key components of a basin wide a flood control plan implemented to protect communities and agricultural areas within the floodplain. Levees are used in conjunction with reservoirs, floodways, control structures, and various channel modification activities to reduce and control the extent and duration of flooding.

The design elevation of levees is based on containing a design discharge, generally for a short period of time. The levee cross section is generally designed as a trapezoid, with an access road running along the levee crown. To control seepage, a long, tapering berm may be extended on the landside of the levee. Fill material for levees is generally obtained locally from borrow areas adjacent to the riverside of the embankment. Although the local materials may not be ideally suitable for construction, economic necessity dictates its use. Less than ideal materials can be compensated for by constructing larger levee sections.

4.2.5.1 Hydraulic Effects

Levees can confine river flows to a narrower cross section, thus higher stages and discharge result during flood flows. If levees are not set back from the main channel, the hydraulic connectivity of the river is lost with the floodplain, thus confining flows and putting more energy into flow. A study reported by Schumm (1977) estimated that levees and dikes on the middle Mississippi River had increased the stage for a discharge of 800,000-900,000 cfs by approximately 10 ft at St. Louis, Missouri.

On un-leveed streams, flood flows spread out over the floodplain. The floodplain acts as storage for the additional flows. The construction of levees decreases the floodplain storage, thus increasing the peak discharge.

Channel instabilities may arise from leveed streams because degradation of the bed and banks may occur. Debate continues on the effect of levees on the Mississippi River. Aggradation may occur due to the increased sediment load in the main channel and the lack of available floodplain sediment storage. The precise response is complex and is a function of the width of levees, the effects on duration of flows, and other factors.

The Midwest flood of 1993 initiated efforts to define a long term, nationwide approach to floodplain management. The results of this effort are summarized in a document commonly referred to as the Galloway report (IFMRC, 1994). It presents an overview of floodplain management, current risks, and the application of structural measures such as levees to minimize flood impacts.

Seepage is a major problem with levees during high water. When water is contained on one side, a head differential exists across the levee. This tends to force water through the porous soil, eventually seeping out to the landward side of the levee. This seepage carries both fine and coarse particles through the levee. This internal erosion of the levees can lead to piping through the levee and catastrophic failure. To prevent excessive seepage, impervious barrier materials such as clay can be built into the levee. Flows from tributaries that are cut off from the river system due to levees must be addressed to prevent flooding on the landward side of the levee. Pumping stations can be applied to divert tributary flows.

4.2.5.2 Environmental Effects

Levees act as a barrier for overbank flows. On un-leveed streams, flows periodically flow onto the floodplain depositing sediment, flushing riparian aquatic environments, and generally providing valuable habitat for aquatic organisms and waterfowl. The flora and fauna are adapted to periodic flooding and the unique environment that it creates. Confining stream flows within a levee system creates a dryer environment on the landside of the levee system and a wetter environment on the stream side. The dryer environment results in changes in both flora and fauna that occupy the floodplain. Studies indicate that after a levee systems are constructed, upland trees and vegetation colonize the floodplain. The lands between the levee and the stream bank will experience more prolonged flooding with more extreme fluctuations in water level. This may inhibit the growth of ground cover, thus reducing the available habitat for ground-dwelling mammals (Fredrickson, 1979). For economical considerations, material used to construct the levees generally are excavated from areas within the floodplain, resulting in vegetation removal and loss of the habitat. The flat slopes used for levees in rural areas require large land requirements for the embankments and berms.

4.2.5.3 Remedial Practices

To offset changes in riparian habitat, consideration is being given to the habitat provided by the levees themselves and the adjacent borrow pits. Traditionally, the vegetation on levees is kept to a minimum. Management of vegetation on levees was investigated on a project along the Sacramento River (Davis *et al*., 1967). The results of the study indicated that with proper maintenance, certain species of shrubs and plants could be allowed to grow without affecting the integrity of the levee. Additionally, the study showed that the cost of maintaining vegetation on the levee was roughly twice the cost of traditional levee maintenance (no vegetation), and that vegetation on levees provides the habitat for burrowing animals that must be controlled. Borrow pits remaining from levee construction can serve as valuable aquatic habitat. Normally, the pits will fill with rain water or groundwater after construction. Riverside borrow pits will exchange water with the river system, thus recharging the pit with fish and other aquatic organisms. Thus borrow pits partially compensate for the loss of aquatic habitat in the floodplain. Additionally, siting levees further from the channel will conserve wetland environments between the levee and the river.

4.2.5.4 Operation and Maintenance of Levees

Levees must be periodically inspected and maintained to provide the designed degree of flood protection. Conditions affecting the integrity of the levee include erosion of the banks, seepage, and damage from burrowing animals. Vegetation planted on the levees for aesthetic reasons should be well maintained. Other vegetation that may affect the integrity of the levee should be removed.

4.2.6 DIVERSION CHANNELS

Diversion channels are constructed to divert waters from the main channel for purposes such as flood control, municipal water supply, and irrigation. A type of diversion channel used for flood control is a flood bypass channel or floodway. It is a separate channel into which flood waters are directed to lessen the impact of flooding on the main river system. Diversion channels on large river systems such as the Mississippi River can consist of adjacent low-lying areas or old river courses. Control structures may be located at the head of the diversion channel to divert flows during periods of high water and return flows during low water. Some diversion channels bypass the flood flows into an adjacent waterway, while others return the flows back into the same stream a distance downstream from the point of the diversion. Diversion channels are often used in urban areas where it is not possible to widen the existing channel due to development. Diversion channels may be used to provide a means of diverting floodwater across the neck of a meander or series of meanders (Acheson, 1968). Major design considerations for diversion channels include: 1) determining if the channel should convey partial or all flows 2) design of appropriate controls 3) sizing of the channel to convey the design discharge and 4) design to reduce maintenance (Nunnally, 1985). To be effective in reducing the flood stage, the distance between the point of diversion and point of return to the main channel must be of sufficient length to prevent backwater effects. Additionally, it is essential to consider potential morphologic effects on both the main channel and receiving channel.

4.2.6.1 Hydraulic Effects

According to Nunnally and Shields (1985), diversion channels generally have steeper slopes than the main channel. This can lead to stability problems such as erosion of the channel bed and banks. The bed of tributary channels may be higher than that of the floodway channel, and bed degradation may migrate upstream of the tributary, resulting in excessive sediment transport and deposition in the floodway. Methods to mitigate channel instability such as grade control, channel lining, and bank stabilization may be required on diversion projects.

Additionally, diversion flows can have an adverse impact on the main channel. From Lane's Balance, it can be seen that reducing the river flow in the main channel due to a diversion, with the slope and particle size remaining constant, will result in a decrease in sediment transport capability, thus aggradation could occur in the channel between the point of the diversion and the point of re-entry. If too much bed material is diverted, the sediment transport capability of the stream may increase, thus accelerating channel instability. Flow returning to the main channel from a diversion can also result in accelerated erosion of the channel and banks. Vanoni (1977) reported that in Alkali Creek in Wyoming, flow returning to the main channel from a diversion resulted in bed erosion. The channel eroded down to an armored layer of large gravel and cobbles, after which the banks began to erode, resulting in the implementation of bank stabilization measures. It is essential that a detailed geomorphic and sediment transport analysis be conducted at the design stage of a diversion project to plan for potential problems.

4.2.6.2 Environmental Effects

It is environmentally beneficial to use diversion channels as an alternative to modifying the main channel to convey flood flows. The original stream substrate and meanders are maintained, as well as in-stream cover and riparian vegetation. If it is designed only for periodic flood flows, the diversion channel can have multiple benefits such as an urban greenbelt, recreation, pasture for grazing, and a wildlife food source (Little, 1973). If the invert of the diversion channel is too low, it will convey both low and high flows, thus continually staying wet. This will inhibit grass growth and increase the possibility of erosion of the substrate. If adjacent low-lying areas or old abandoned river courses are used for diversion purposes, some terrestrial habitat may be lost or converted to a wetland habitat.

4.2.6.3 Remedial Practices

The diversion system must be carefully designed and constructed to prevent channel instability in the main channel and the diversion channel. Channel design must take into account the design flows and sediment transport to insure bed and bank stability. The hydraulic design of diversion channels can be accomplished with standard hydrology and hydraulics analysis techniques, while determinations of sediment transport through the diversion are much more difficult. Because the floodway invert is higher than that of the main channel, there is a tendency for the channel to become unstable and degrade. Grade control structures may be necessary on the downstream end of the floodway to prevent upstream migration of bed degradation, and on any perched tributaries that are hydraulically connected to the diversion channel.

4.2.6.4 Operation and Maintenance of Floodway Projects

Diversion channels that have a seasonal covering of grass will require maintenance, and should be designed with sideslopes conducive to mechanical mowing. Efforts should be taken to insure that the channel invert is constructed above the seasonal high water table to prevent excessive growth of aquatic vegetation that interferes with maintenance.

4.2.7 DAMS

Impoundments are constructed for multiple uses. In canalization projects, dams are constructed along with locks for navigation purposes. Dams and associated reservoirs are built on rivers primarily for flood control, with secondary functions such as recreation, water supply, and power generation. Sediment retention dams are utilized as flow control to reduce sediment loading to downstream areas (USACE, GDM-54, 1990a). One or more dams are constructed in the upper watershed to trap sediments and thus reduce bed material load downstream. Additionally, dams reduce the sediment load by changing the flow duration curve for the stream. Controlled releases through the dam reduce the flood peaks and subsequently reduce the sediment load downstream. Peterson (1986) describes the social and environmental impacts of dams on a number of river basin projects. The beneficial uses for which a dollar

value can be assigned were for flood control, hydropower generation, irrigation, and recreation. For the Columbia River dam projects the adverse environmental impacts were primarily due to the dams blocking the salmon migration routes. Because of the multipurpose nature of some dams, it is difficult to optimize the beneficial aspects of each use. From the flood control viewpoint, it is necessary to reduce flood peaks downstream. This practice may result in inadequate flows for power generation and disrupt fish spawning. Dams change the flow and sediment transport characteristics of the river. The back water extends upstream of the dam, acting as a sediment retention basin. Regulated flows through the dam along with reduced sediment transport below the dam may affect downstream channel stability.

4.2.7.1 Hydraulic Effects

The primary effect of dams on system stability is to reduce peak discharges and sediment supply to the downstream channel. Upstream effects of a dam and associated reservoir include delta formation, gradual raising of stream levels in the backwater zone, and a more pronounced meandering (USACE, EM 1110-2-1418, 1994). Downstream effects result from flow control through the dam and retention of sediment. A reduction in peak discharge often reduces bank instability downstream by inducing deposition at the channel margin in the form of berms. The channel adapts to a lower channel forming discharge by shrinking. Reducing peak discharge and lowering the flowlines in the downstream channel may also induce tributary instability by lowering their effective base level. Channel degradation in the form of a head cut advances up the tributaries and ultimately increases the sediment supply to the main river. However, reducing the sediment supply to the stream through reservoir retention also often induces channel degradation downstream, which can actually lead to mass instability of the banks by increasing bank heights. This may trigger a reversal of main channel response and lead to eventual aggradation due to increased sediment supply from tributaries (Biedenharn, 1983). System response to flow control and sediment retention aspects of dams are very complex and cannot be easily predicted or generalized. Factors affecting channel response:

a. Magnitude and frequency of flow duration;
b. Degree of sediment retention;
c. Downstream controls such as geologic outcrops, man-made structures, armor layers and backwater from another lake or river;
d. Reduced sediment transport capacity of the channel as a result of slope reduction due to channel degradation;
e. Sediment input from tributaries and bed and bank erosion;
f. Vegetation and vegetative encroachment; and
g. Tributary response.

4.2.7.2 Environmental Effects

The construction of dams results in a decrease in terrestrial habitat through backwater flooding. However, case studies of dams on selected river basins presented by Peterson (1986) indicate that reservoirs have had a lesser impact on wildlife than urbanization and agriculture. Green and Eiker (1983)

reported that while the reservoirs on the Columbia River basin did decrease the habitat for some mammals, waterfowl habitats increased. Babcock (1980) reported that on the Arkansas River Navigation Project, the environmental quality actually improved due to construction of the project. The water quality improved with a reduction in suspended solids. Dam outflows generally are at a lower temperature than existing channel flows. The lower water temperature may be suitable for specific species of fish such as trout and deleterious for native warm water fish populations, and the fishery diversity may be permanently altered. Aquatic and terrestrial habitats are impacted by a reduction of flushing flows through the dam. In periods of low flow through the dam, fish and other aquatic organisms that depend on higher flows for food and habitat are affected. Terrestrial habitat along the stream that experienced frequent overbank flows in pre-dam conditions may be dry for prolonged periods of time, thus potentially displacing wildlife dependent on a more wet environment. Additionally, flows through the dam will be based on needs such as hydropower, flood control, and recreation. This will result in a change in the channel forming discharge that will alter channel morphology and subsequent habitat features. Throughout the country, such as the Northwest, fish passage around dams is a serious environmental concern. Dams block migrating fish such as salmon from completing spawning runs.

4.2.7.3 Remedial Measures

The construction of dams can adversely impact downstream channel stability. Channel and streambank remediation techniques may be required to reduce erosion and deposition of sediments resulting from fluctuating flows and reduced sediment transport through the dam. Changes in dam operating procedures can be made to accommodate environmental needs. Periodic flushing flows can be released to enhance downstream aquatic and terrestrial habitats. Fishways or fish ladders can be used to allow migrating fish to bypass dams. On the lower Snake River in Washington, salmon are bypassed around dams using barges. In some cases, dam removal is advocated to restore a rivers natural and recreational value.

4.2.7.4 Operation and Maintenance of Dam Projects

Dams require frequent inspections to insure structural integrity. Grade control structures and streambank stability projects constructed to remediate channel and bank erosion require periodic inspection to insure proper operation.

4.2.8 FLOW TRAINING STRUCTURES - DIKES

Dikes are free standing structures of stone, pile clusters, or pilings with stone fill placed within waterways either parallel or transverse to the channel, and are generally constructed to constrict the channel at a specific location for the purpose of concentrating flow in a narrower, deeper channel. The reduced cross sectional area results in an increase in flow velocities thus increasing the sediment transport capability of the stream. In navigable rivers the decrease in shoaling reduces dredging requirements. Dikes have been

used extensively on the Lower Mississippi River to maintain navigation channels, and can be used in conjunction with other measures such as floodways, cutoffs, bank protection and levees to aid in flood control, maintain navigation, and stabilize river systems. Additional applications include cutting off side channels and chutes, concentrate a braided river into a single channel, realigning a river reach, and streambank protection.

A variety of materials can be used to construct dikes. Stone dikes and pile dikes are the most common type in use, but soft dikes consisting of sand filled geotextile containers have been used successfully on the lower Mississippi River. Dikes may be constructed either parallel or perpendicular to the flow. Spur dikes, which are sometimes referred to as transverse or cross dikes, are the most common types of dikes used on major streams (Shields and Palermo, 1982). Dikes are generally constructed in groups perpendicular to the flow, extending outward from the bank toward the center of the channel. Spacing between dikes in a dike field is generally a function of the location of the next dike downstream (Peterson, 1986). Longitudinal dikes extend downstream and parallel to the flow. The primary purpose is for reducing the curvature of sharp bends and provides erosion protection for the adjacent bank.

L-head dikes consist of both a section perpendicular to the flow extending from the bank, and a section parallel to the flow extending downstream from the end of the perpendicular section. L-head dikes are designed to reduce sedimentation behind the dike and can be used to reduced sedimentation in specific areas such as harbor entrances.

4.2.8.1 Hydraulic Effects

Dikes are designed and constructed to confine flows in a narrow channel and induce an increase in sediment transport through the channel. Depending on design, dikes can affect the flow in a number of ways. For example, spur dikes, which extend perpendicular to the flow, are used to constrict the flow and concentrate the flow within the constricted reach. Longitudinal dikes are arranged downstream and parallel to the flow, and are used to reduce the curvature of sharp bends, develop stable channel alignments, and provide erosion protection for the adjacent bank. Because of the increased velocities, localized scour and undercutting occurs at the end of the transverse dike. Incorporation of design criteria such as improved profile slope and dike angle can reduce the effects of scour. At low water, sediment deposition occurs in the slack water between dikes.

4.2.8.2 Environmental Effects

Shields and Palermo (1982) report work by Thackston and Sneed (1980) and Johnson *et al.* (1974) which identified three areas of environmental impacts due to dike fields: 1) impacts associated with dike construction, 2) changes in water surface area and aquatic habitats, and 3) increased water-level fluctuation. Because the majority of dike construction occurs in depositional zones near the bank, some benthic habitat is lost during construction. Additionally, construction techniques may temporarily increase localized turbidity. Dikes increase the habitat diversity. The areas between the stones and downstream of the dike provide feeding and resting areas for fish. Slack water between dikes provides additional aquatic habitat unless excessive sedimentation occurs.

A gradual build-up of sediment occurs in the slack-water areas between dikes during high flows. At low flows, the shoals may be out of water, thus allowing vegetation such as willows to colonize the area. During high flow events, the increased vegetation effectively increases the roughness thus further encouraging sediment deposition. This results in a decrease in aquatic habitat and an increase in terrestrial habitat. Brookes (1988) reports a study by Morris *et al*. (1968) that reported that the construction of pile dikes on the Missouri River in Nebraska reduced the width from 720 to 240 meters and reduced benthic habitat by approximately 67 percent. Habitat diversity may be reduced by stabilizing the stream with dikes.

4.2.8.3 Remedial Measures

Although a reduction in sedimentation in dike fields can be achieved by varying the length and height of dikes, constriction gaps or notches in dikes are presently the most widely used environmental restoration method (Shields, 1983). Notched dikes are used to mitigate the loss of aquatic habitat due to sedimentation on the downstream side of dikes. Stone is removed from the dike to a specific width and depth to create a gap allowing flow to pass through the dike. The flow through the gap induces scour that removes sediment deposits and restores aquatic habitat. The notch width, shape, and depth design can varied to provide varying degrees of habitat restoration. Notch openings should be adequate to provide the necessary effect of creating habitat without causing excessive erosion or deposition.

The Missouri River Division of the Corps of Engineers has used notched dikes to restore aquatic habitat on the Missouri River (Shields and Palermo, 1982). Small gaps in the Missouri River dikes were observed to produce small chutes and submerged bars behind the dikes, whereas large openings created open-water habitat.

4.2.8.4 Operation and Maintenance of Dikes

Dikes, like other in-stream structures, require inspection to insure proper operation.

4.2.9 GRADE CONTROL

The most common method of establishing grade control is the construction of in-channel grade control structures. There are basically two types of grade control structures. One type of structure is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the degradational zone. This is somewhat analogous to locally increasing the size of the bed material. Lanes's relation would illustrate the situation by $QS^+ \alpha Q_s D_{50}^+$, where the increased slope (S^+) of the degradational reach would be offset be an increase in the bed material size (D_{50}^+). This is referred to as a bed control structure. Sills are placed across the channel at or just above the bed elevation to control scour. Materials such as concrete rubble, stone, or locally available non-erodible materials can be used. The sill acts as a hard point in the channel that resists erosion, thus stabilizing the bed. Channels may be completely stabilized by lining the channel with non-erodible material such as concrete or stone. This is a

more expensive alternative, but it may be necessary in urban areas where land costs are high, thus narrow channels with steep side slopes are desirable.

The second type of grade control structure is designed to function by reducing the energy slope along the degradational zone to the point that the stream is no longer capable of scouring the bed ($QS^{-} \alpha$ Q_s D_{50}), which requires establishing a hydraulic control at the structure. Examples of hydraulic control structures are weirs and drop structures. Weirs are placed across the channel to control the water level thus controlling the stream energy gradient. For large discharges or significant changes in bed elevation, drop structures are employed. Drop structures are designed to limit and stabilize channel bed slope by means of a vertical drop.

4.2.9.1 Hydraulic Effects

The function of hydraulic grade control structures is to reduce the energy slope along the degradational zone, thus reducing the ability of the river to scour the bed. This results in a backwater above the structure and a subsequent lowering of the velocity. These areas typically are more conducive to sedimentation, thus the affected reach is transformed from degradational (erosive) to aggradational (depositional). This sediment trapping affect along with the desired affect of reducing bed erosion will deprive downstream reaches of sediment, thus possible affecting downstream stability. Grade control structures can affect the flood potential of the stream. Hydraulic grade control structures are often designed to be hydraulically submerged at flows less than bankfull so that the frequency of overbank flooding is not affected. However, if the structure exerts control through a wider range of flows including overbank, then the frequency and duration of overbank flows may be impacted. Another factor that must be considered when siting grade control structures is the safe return of overbank flows into the channel. This is particularly a problem when the flows are out of bank upstream of the structure but still within bank downstream. The resulting head differential can cause damage to the structure as well as severe erosion of the channel banks depending upon where the flow re-enters the channel.

4.2.9.2 Environmental Effects

Grade control structures can provide direct environmental benefits to a stream. A study was conducted by Cooper and Knight (1987) on fisheries resources below natural scour holes and man-made pools below grade control structures in north Mississippi. The study results conclude that although there was a greater species diversity in natural pools, there was increased growth of game fish and a larger percentage of harvestable-size fish in the man-made pools. Shields *et al*. (1990) reported that the physical aquatic habitat diversity was higher in stabilized reaches of Twentymile Creek, Mississippi than in reaches without grade control structures. Jackson (1974) documented the use of gabion grade control structures to stabilize a high-gradient trout stream in New York. She observed that following construction of a series of bed sills, there was a significant increase in the density of trout. The most serious negative environmental impact of grade control structures is the obstruction to fish passage. In cases where drop heights are small, fish are able to migrate upstream past a structure during high flows (Cooper and Knight, 1987). However,

where structures are impassable, openings, fish ladders or other passageways must be incorporated into the structure design to allow fish migration.

4.2.9.3 Remedial Measures

When designing hydraulic control structures, overbank flooding concerns must be addressed. The potential for causing overbank flooding may be the limiting factor with respect to the height and amount of constriction at the structure. If the structure exerts control through a wider range of flows including overbank, then the frequency and duration of overbank flows may be impacted. The impacts must be quantified and appropriate provisions such as acquiring flowage easements or modifying structure plans should be implemented. The safe return of overbank flows must be considered when siting the structure. One method is to design the structure to be submerged below the top bank elevation, thereby reducing the potential for a head differential to develop over the structure during overbank flows. Direct means of controlling overbank flows include constructing an earthen dike or berm extending from the structure to the valley walls to prevent flows from passing around the structure and constructing an auxiliary high flow structure that will pass overbank flows to a specified downstream location.

4.2.9.4 Operation and Maintenance of Grade Control Structures

Monitoring and maintenance of grade control structures is essential to ensure adequate performance. Because of the dynamic nature of streams, lack of monitoring and required maintenance can result in complete failure of expensive installations. Monitoring should include upstream and downstream conditions that may have future impacts upon the project. Examples are: 1) changes in upstream channel alignment may threaten bank stabilization works downstream 2) channelization work may induce degradation upstream and may change hydraulic and geomorphic conditions downstream and 3) significant changes in operating procedures of reservoirs upstream of the project site or significant land use changes may change hydraulic and geotechnical parameters at the site. A monitoring program should consist of site inspections, site surveys, geomorphic observations, hydrologic and hydraulic data, geotechnical data, and environmental aspects.

4.2.10 BANK STABILIZATION

As discussed in Chapter 3 of this manual, the instability and subsequent failure of stream banks commonly result from a combination of hydraulic, geomorphic, and geotechnical factors. Scour occuring on the outside of channel bends increases bank heights and subsequently leads to bank failures. The terms streambank erosion and streambank failure are often used to describe the removal of bank material (Biedenharn *et al.*, 1997). Erosion generally refers to the hydraulic process where individual soil particles at the banks surface are carried away by the tractive force of the flowing water. Therefore, the erosive forces are generally greater at higher flows. The primary erosion processes are parallel flow, impinging flow, piping, freeze thaw, sheet erosion, rilling and gullying, wind waves, and vessel forces. Streambank

failure differs from erosion in which a relatively large section of bank fails and slides into the channel. Streambank failure is often considered to be a geotechnical process. A geotechnical failure involves the movement of a relatively large and possibly intact segment of soil. There are two distinct classes of bank failure: the slow moving creep and the catastrophic shear failure. The slow moving creep failure occurs over long periods of time, whereas the catastrophic shear failure occurs instantaneously.

Channel instability can ultimately result in system-wide bank instability. As channel degradation proceeds through a system, the channel bank heights and angles are increased, which reduces the bank stability with respect to mass failures under gravity. If degradation continues, eventually the banks become unstable and fall. Bank failures may no longer be localized in bendways, but rather may also be occurring along both banks in straight reaches on a system-wide basis. Fluctuating flows through channels and localized runoff can also contribute to accelerated erosion of the banks.

System-wide instability is treated with channel stabilizing methods described above. Localized bank erosion and failure is treated with a variety of methods designed to either directly or indirectly protect the bank (Shields and Palermo, 1982). Bank stabilization projects address local problems such as meander migration and constricted reaches and are not a remedy for system instability. Direct bank protection methods are placed in contact with the bank to prevent erosion. Indirect protection methods are designed to deflect flows from the affected area or reduce turbulence and encourage sediment deposition. Example of direct methods are stone riprap, trench fill revetment, concrete paving, articulated concrete mattresses, and vegetation. From an environmental viewpoint, vegetation is the preferred treatment when hydraulic conditions allow its use. Woody vegetation is usually restricted to banks, but grass linings may be used if properly maintained and not exposed to excessive velocities (Nunnally and Shields, 1985). Indirect methods include dikes, fences, and jacks. More detailed information concerning design and placement of channel and bank stabilization methods is provided in the *WES Stream Investigation and Streambank Stabilization Handbook* (Biedenharn *et al.*, 1997).

The primary purpose of reservoir construction is usually flood control or water supply, but reservoirs may also be designed specifically to induce channel stability and subsequently stabilize banks. The effect of reservoirs is to reduce peak discharges and sediment supply to the downstream channel. A reduction in peak discharge often reduces bank instability by inducing deposition at the channel margin in the form of berms. In effect the channel adapts to a lower effective or dominant discharge by shrinking. Bank failure upstream of reservoir impoundments will be decreased by the reduction in flow velocities and bank shear stresses for the length of the channel affected by the impoundment.

4.2.10.1 Hydraulic Effects

Indirect bank stabilization methods act to deflect flows from affected areas or reduce current velocities adjacent to banks. After eroding banks are stabilized, the sediment discharge is reduced in the system. If the reduction of sediment discharge is significant, the system may adjust by eroding and degrading the channel bed. The operation of reservoirs to accomplish bank stability by reducing peak discharge and lowering the flowlines in the downstream channel may result in tributary instability be lowering their effective base level. Additionally, the reduction of sediment supply downstream from reservoirs may induce channel degradation downstream. This can result in increased bank heights and bank instability.

4.2.10.2 Environmental Effects

Direct methods of streambank protection initially involve some bank preparation and removal of vegetation. This initial adverse impact on the riparian ecology is offset by the benefit of halting the existing erosion. Bank protection can increase habitat diversity if the bank is re-vegetated with environmentally beneficial plants as part of the bank protection scheme. Extensive streambank protection works can result in a reduction of channel migration, which reduces habitat diversity.

4.2.10.3 Remedial Measures

Currently, environmentally compatible methods of stream bank protection are based on extensive use of vegetation, particularly used in combination with structural applications. Allen (1978) describes the use of plants to control erosion of streambanks, reservoir shorelines, and other areas. Shields and Palermo (1982) indicate field studies were conducted on the Missouri, Sacramento, Willamette, and Lower Mississippi Rivers on the environmental effects of bank protection projects and methodologies to reduce adverse environmental effects. Demonstration projects were conducted in the Ohio and Yazoo River Basins for testing various combinations of vegetation and structure. The reduction in habitat due to paved channels can be alleviated by the use of riprap as a lining, with the voids between the riprap filled with stream gravel.

4.2.10.4 Operation and Maintenance of Channel and Bank Stabilization Projects

As with all in channel structural improvements, periodic inspections are required to insure that the project functions as intended. For direct methods of bank protection, inspections should be conducted to insure that surface armor such as riprap remains in-place and has not been displaced by high discharge events. The channel should be periodically inspected for signs of instability that could cause future maintenance problems.

4.2.11 CHANNEL RESTORATION

After channel modification projects have been constructed, adverse environmental impacts can be mitigated through channel enhancements and restorative methods. The primary impact on stream ecology from channel modification is the reduction in habitat diversity. Channel clean out, enlargement, or straightening practices may result in removal of the existing substrate, pool and riffle areas, and riparian vegetation and canopy. The resulting effect on the aquatic environment is a reduction in the diversity of aquatic life as well as population densities. The goal for restoration activities is to accelerate biological recovery of the stream through the use of various techniques and methodologies without impacting the stability of the stream.

In-stream structures are used to increase habitat diversity by altering flows, changing channel morphology and substrate, and providing cover. Artificial structure such as boulders, boulder clusters, or concrete can be randomly placed in the channel to provide zones of reduced velocity and scour holes downstream of the boulder. Sills can be constructed across the waterway to create pools above and scour holes below the structure. Sediment scoured from below the sill may redeposit some distance below to form a riffle area. A series of sills installed in the stream will develop a pool and riffle sequence that is highly desirable for providing feeding and resting areas for fish and aquatic organisms.

Channel modification usually results in poorly sorted, finer, less stable bed material (Shields and Palermo, 1982). A study reported by Arner *et al*. (1976) indicated that fine, poorly sorted sediments in a modified segment of the Luxapalila River, Mississippi resulted in a reduction in the quality and quantity of aquatic organisms. The replacement of natural bed sediments following project completion may speed the biological recovery. This is more successful when well sorted gravels replace unsorted sediments (Hjorth and Tryk, 1984). Substrate reinstatement was used to speed the biological recovery of a stream relocated to allow for coal mining. Gore and Johnson (1980) reported that material excavated from a coal mining operation was used to line the relocated channel with layers of topsoil, gravel, and cobbles. Benthic organism populations were rapidly established in the channel by colonization from undisturbed stream reaches.

Low flows in enlarged channels may be too shallow to support fish and be devoid of pools. Shallow channels can be excavated within modified channels to convey low flows and provide the necessary depth for supporting fish and other aquatic organisms. A study conducted by McCall and Knox (1978) described the environmental benefit of utilizing a low flow notch design in a modified channel for Rock Creek in north-central Indiana. One year after completion, 23 species of fish were found in the low flow channel, compared to 16 species collected from the natural channel upstream of the low flow channel section.

Grade control structures such as weirs and drop structures obstruct fish movement and migration in the channel. Additionally, culvert and shallow channel sections in which the flow is too slow or swift impede the natural movement of fish. Fishways or fish ladders are designed to allow fish to either by-pass or pass through channel obstructions.

In some cases, it may be justified to restore the former sinuosity to the modified stream. This action is taken assuming that the engineering function for which the channel was originally modified is either no longer required or will not be impacted, and that no major watershed changes have occurred since initial straightening that would disrupt the equilibrium of the restored channel. In Southern Denmark, a new channel was constructed to replace an 800 meter section of severely degraded channel (Brookes, 1987). The original sinuosity was determined from historical maps, comparison of other neighboring streams, and field reconnaissance of the watershed. Native grasses and woody vegetation were planted for stabilization, with riprap used for bend stabilization before vegetation became established. The new sinuous channel restored morphologic and hydrologic diversity, with colonization by a number of flora and fauna. In West Germany, Glitz (1983) described the restoration of the sinuosity of the Wandse river in Hamburg-Rahlstedt, a lowland river about 1.5 meters in width. A partial restoration was performed assuming that the stream would eventually adjust naturally. A survey conducted two years later indicated that pool and riffle formations were limited, probably due to the low energy of the stream.

Management practices may be implemented to preserve the morphological and ecological aspects of the channel without modifying the existing channel to accomplish engineering goals. The concept of floodplain corridors provides sufficient land area on both sides of the stream to allow for natural migration of bends and general channel shifting across the floodplain. This allows the natural formation of habitat enhancement features such as pools, riffles, and point bars. Future watershed planning and management activities are possible with the channel confined to a fixed position on the floodplain.

4.2.11.1 Hydraulic Effects

The use of channel restoration techniques to enhance stream ecology is growing. Many of the restorative methods have a limited influence on hydraulics of the channel. The use of artificial structures and sills to create a pool and riffle habitat do not have a significant impact on stream hydraulics, particularly at high flows for which the structures are inundated and no longer function as intended (Brookes, 1988). However, the use of in-channel vegetation can significantly increase the roughness and consequently reduce the discharge capacity of the stream. Wilson (1973) determined that vegetation such as willows and shrubs can reduce the discharge capacity up to 50 percent after only one year of growth. The use of vegetation within stream channels for purposes such as restoration or bank protection requires a thorough hydraulic and sediment transport analysis during the project design phase. Low energy stream systems with moderate flows and low sediment transport are more amenable to vegetative projects. The survival of vegetation in high energy channels with high peak flows and substantial sediment transport is questionable. To insure a successful project, a multi-disciplined team consisting of biologists and hydraulic engineers is recommended.

4.2.11.2 Environmental Effects

Activities and practices implemented for stream restoration should have a positive impact on stream ecology. The introduction of artificial habitats into modified channels provides the diversity necessary to support a wide variety of aquatic organisms and fish in otherwise unsuitable habitat. Modifications to channel morphology in terms of restoring stream meander or sinuosity must be carefully planned to avoid creating channel stability problems. The examples presented above on restoring sinuosity were for low energy channels that under natural conditions do not actively migrate. Additionally, if watershed changes occur that alter the sediment and water discharge of the original watershed, attempts to alter channel morphology may disrupt the equilibrium of the restored channel.

4.2.11.3 Operation and Maintenance of Channel Restoration Projects

The incorporation of artificial structure such as boulders and sills into a modified channel design will required a periodic inspection plan to insure that the structures remain effective. Habitat enhancement features should never be placed in an unstable channel subjected to cyclical sediment erosion and deposition processes. Periodic channel inspections upstream and downstream of the project are required to evaluate system stability and determine potential future project maintenance problems.

4.3 SUMMARY

Channels are modified from their natural state for beneficial uses such as flood control, navigation, and water supply. Additionally, channel modifications are required to treat the impacts of channel instability (bed degradation and excessive sedimentation) resulting from changes in channel and basin sediment and water discharge capacities. Modifications can result in adverse impacts to channel and riparian ecology. The primary environmental impact of channel modification is the reduction of habitat diversity. Straight, shallow channels with homogeneous substrates that are devoid of vertical relief, such as pools and riffles, do not provide the necessary food, cover, and resting areas for fish and other aquatic organisms. The destruction or elimination of riparian and riparian vegetation reduces the habitat for mammals and birds, eliminates plants that provide necessary shade for the stream, and accelerates the erosion of streambanks. The goal of this chapter was to present general descriptions of channel modification projects, activities, and practices along with associated impacts on channel stability and ecology. More specific information on design and implementation of environmental restoration practices can be obtained from the references cited in this chapter, most notably the references of Shields and Palermo (1982), Nunnally and Shields (1985), Shields (1983), and Brookes (1988).

Alluvial streams left in a natural state will strive to attain an equilibrium condition for which the energy available in the stream (water discharge and stream gradient) is proportional to the energy required to transport a given sediment quantity and size. This qualitative relationship as described by Lane (1947) provides the basis for qualitatively describing channel and bank stability problems associated with channel modification activities. Each of the examples of channel modification discussed in the above sections can

impact channel stability by changing one or more of the key variables responsible for maintaining channel stability. Before any habitat restoration or environmental enhancements can be implemented on a modified stream, channel stability issues must be addressed through consideration of channel rehabilitation methods. Channel stability evaluations should include the entire basin and not only the affected modified reach.

CHAPTER 5

FUNDAMENTALS OF ENGINEERING DESIGN

The primary focus of Chapter 5 is the fundamentals of engineering design that were used in developing the preliminary design of stabilization measures. Whenever appropriate, example computations or methodologies will be presented. In particular, methods developed under the Demonstration Erosion Control (DEC) Project will be used as an example. The DEC Project, administered by the U.S. Army Corps of Engineers, provides for the development of a system for control of sediment, erosion, and flooding in the hill areas of the Yazoo Basin in Northern Mississippi.

5.1 BACKGROUND INVESTIGATIONS

The background investigations section describes research that develops a characterization of the basin parameters used to identify main river processes and controls that dominate flow conditions. The background research examines the climate, geology, geography, and hydrology of the basin and determines the relationships and effects of these parameters on the stream. In addition, these investigations should identify any large-scale instability or disturbance that may be directly affecting the stream stability. An integral part of the background investigations is the research on physical indicators and records of past behaviors. The historical record should be examined, if available, and used to investigate past adjustments as indicators of future adjustment. Local river surveys from area agencies, local archives, and county government survey departments are excellent places to begin. Table 5.1 suggests some sources of historical information.

The types of information that can be obtained from these sources are channel and reservoir surveys, flood history, watershed workplans, bridge plans and surveys, watershed erosion information, geological data, Drainage District records, land use records, existing erosion control features, land ownership information, historical sediment yield data, and aerial photography. Past lateral and vertical migration patterns and extreme flow events can be examined from historic maps, aerial photographs, historic photos and descriptions, botanical records and paleostage indicators. Documentation of previous land use changes can sometimes be correlated with and used to examine past stream response.

Table 5.1 Suggested Sources of Historical Information (USACE, 1994)

Previous studies and reports: Corps of Engineers, U.S. Bureau of Reclamation, consultants
U.S. Geological Survey Quadrangle Sheets: old and new series
Aerial photographs
Topographic maps by the Army Map Service and others
FEMA Flood Insurance Studies
County maps and city plots
Offices of county, state, highway, and railroad engineers
Local newspapers
Older inhabitants, especially farmers
U.S. Geological Survey: gage histories and descriptions, gaging notes, rating curves through period of record; water supply papers; provisional discharge records
National Weather Service: storm and flood records
Municipal water and power plants: gage records
Irrigation and drainage districts: gage records

5.1.1 GEOLOGY

Geological considerations include valley slope, description of the predominant material in which the channel is formed, tectonic activity, and the effects of large-scale man-made projects. Valley slope affects several characteristics. The slope of the valley can be determined from field surveys and from topographic maps. Soil erosion in the basin depends, to a certain extent, on the valley slope. Steep valley slopes increase the erosion capacity of the overland flow which can lead to increased sediment yields. Discontinuities in the slope can also affect stream pattern and stream sediment carrying capacity.

Classification of the material of which the channel is formed directly affects the erosion resistance. Material properties will influence the susceptibility of the basin to geomorphic and sedimentary processes. A channel incised in bedrock can be considered to be a stable reach that will not migrate significantly and requires little control to keep it from shifting location or pattern. Alluvial streams, in contrast, are those in which the bed and banks are composed of material that has been deposited by the flow. Because the bed and banks of alluvial streams are generally composed of erodible material, the channels are dynamic features that are free to shift position or patterns. One consequence of this characteristic is that alluvial streams readily respond to changes in the basin. Rock outcrop in alluvial streams act as a control and can restrict horizontal and lateral migrations as well as affect the depth of flow. The positions and any obvious effects of outcrop on the flow should be noted.

Although tectonic activity usually occurs at very slow rates and is difficult to quantify, the effects on fluvial processes and evolution can be significant. Tectonic forces such as faulting, folding, or tilting primarily affect river systems through differential changes in gradient. Uplift or subsidence may disrupt the environment or produce a change in hillslopes in the basin and alter the delivery of sediment (Leopold *et al.*, 1964, p. 475). Channel pattern is a sensitive indicator of valley slope change. To maintain a constant

gradient, a stream that is steepened by uplift will increase sinuosity while a reduction in valley gradient will lead to a reduction in sinuosity. A secondary effect of the changing gradient is the change that may occur to the sediment load. Active tectonics can result in a unstable stream, which can be reflected by incision, deposition, bank erosion, meander cutoffs, or change in stream patterns (Gregory *et al.*, 1987, p. 65). A more complete description of the effects of tetonic activity on alluvial rivers can be found in Schumm *et al.* (1982).

The effects of man are the next variable to be examined. Many man-made structures affect the stream in the same manner as geographic and geologic controls. Water diverted through diversion structures can significantly alter discharge in the main channel. Dams and reservoirs affect the sediment budget, depth of flow, total discharge, as well as the hydrograph shape. Bridge abutments, check dams, and other man-made structures can act like constrictions and seriously affect the flow characteristics. The presence of these structures should always be noted in the study reach as well as any obvious effects on the flow regime.

5.1.2 GEOGRAPHY

The geography of the region should be examined. This includes a description of the general location of the site, the basin size and terrain, primary land use, basin habitat and vegetation, and large-scale disturbances or instabilities. The general location and description of the site can be referenced by stream name, township, range and section location, the nearest town or distinguishing landmark, and the county and state in which the reach occurs. If a global positioning system unit is available, a latitude and longitude can be added to location description. It is important to be able to identify the site for future reference. A topographic map can also be used to determine the size and terrain of the drainage basin.

The primary land use should be documented as this variable directly affects the degree of runoff potential, hydrograph shape, sediment yield, and to some extent the amount of human interaction that can be expected with the stream. Land use (Table 5.2) is categorized as urban, rural, agricultural, or conservation. Urban is defined as intensive residential, recreational, commercial, or industrial use. Urban land use is considered significant if greater than 25 percent of the basin contributing to the study reach is urban and is characterized by possible large quantities and high variability of the sediment load. Rural land use is defined as containing small farms and low density residential areas and contribute smaller sediment loads, on average, and experience less variability in the sediment load. Rural land use is considered significant when more than 45 percent of the drainage basin is rural. Agricultural land use refers to areas where crops, orchards, pastures, and forests are being used for production and can contribute large amounts of sediment, depending on the cropping practices employed. Agricultural classification is appropriate for basins in which more than 35 percent of the basin is utilized for production. Conservation land use is defined as no development and can include swamps, grasslands, forests, and lakes and the quantities and variability of sediment are generally both very small. Conservation land use is considered significant if more than 65 percent of basin is undeveloped. In all of the land use classifications, knowledge of the

specific practices being used and the amount of construction underway or planned enables the engineer to give a better estimate of the qualitative effects of the land use in the basin.

Table 5.2 Land Use Classification (after Rundquist, 1975)

Urban	Greater than 25% of drainage basin is urban
Rural	Greater than 45% of drainage basin is rural
Agricultural	Greater than 35% of drainage basin is agricultural
Conservation	Greater than 65% of drainage basin is conservation

The degree of human interaction with the stream can also be estimated from the land use classification. The effects of humans are expected to decrease as development decreases. Fewer people living near the channel may require less flood control and channel control. Urban land use can then be expected to contribute a higher degree of human impacts on a stream while conservation land use will experience the smallest degree of effects from human contact.

5.1.3 SEDIMENT

Sediment moving in a stream can be defined on the basis of the measurement method, the transport mechanism, and the sediment source. Table 5.3 summarizes the three methods of sediment classification.

Table 5.3 Three Methods of Sediment Transport Classification

Measurement Method	Transport Mechanism	Sediment Sources
Unmeasured Load	Bed Load	Bed Material Load
+	+	+
Measured Load	Suspended Load	Wash Load
= Total Load	= Total Load	= Total Load

Typically, stream data is reported as the measured suspended sediment load, and is measured using a cable-suspended sampler that is lowered and raised through the water column. However, the sampler inlet cannot be accurately positioned on the stream bed and a portion of the sediment transported is unmeasured. Therefore, sediment transported can be defined as either measured or unmeasured load.

Sediment can be defined by the type of transport mechanism, either the bed load or the suspended load. The bed load transport mechanism is by particles rolling, skipping, or sliding along or in intermittent contact with the bed. Suspended sediment moves in the water column above the bed and is not in close contact with the bed at all times.

The third basis for definition of sediment transport is by the sediment source. Bed material load is the material that is found in appreciable quantities in the bed. Wash load is finer, and is not of a size found in appreciable quantities in the bed.

5.1.4 MEASURED SEDIMENT DATA - RATING CURVE DEVELOPMENT

The sediment transport capacity of the river is determined by developing a sediment rating curve which defines total sediment load (suspended and bedload) as a function of discharge. An example of the approach used for developing a sediment rating curve is presented below for the DEC Project.

For the DEC Project, suspended sediment samples were collected in a consistent manner by the USGS at each of the ten USGS gaging stations utilized in the present research. Observers collected single, vertically integrated samples three days each week (Monday, Wednesday, and Friday) and supplemented these data during selected storms. Each site is also equipped with a PS-69 automatic point sampler, which is stage activated during storms. USGS personnel collected samples on a biweekly basis and during selected storms, which may be either single or multiple, vertically integrated samples taken at several sections across the stream (Rebich, 1993). The sampling procedures are described by Guy and Norman (1970).

The effort by the USGS is significant, for example from July 1985 through September 1991, about 20,000 suspended-sediment samples were analyzed and reviewed, and data were stored in USGS computer files. Sediment samples were then used to compute daily mean values of suspended sediment concentration and sediment discharge according to standard USGS procedures (Porterfield, 1972).

Sediment rating curves were developed from the observed USGS data, with sediment discharge as a function of water discharge. Figure 5.1 depicts the log-log ($Q_s = aQ^b$) of total suspended sediment discharge as a function of water discharge for Fannegusha Creek. Table 5.4 presents the regression results for ten USGS gaging stations.

Table 5.5 provides similar regression data for the bed material portion of the measured sediment discharge for the gages. Sand is the predominant bed material found in substantial quantities in the shifting portions of the bed for the gage locations. Data were not available for the sand portion of the sediment discharge at Otoucalofa Creek.

Figure 5.2 portrays the regression of the USGS sand fraction data, the measured USGS sand fraction data, and a computed total sand discharge, which was computed using a HEC-2 computation of the hydraulic characteristics of the backwater, and the SAM program (Thomas *et al.*, 1994) using the Brownlie (1981) sediment transport equation for the Abiaca Creek, Site No. 6. Close agreement is apparent between the Brownlie computation of the total bed material load and the regression of the observed USGS sand fraction regression. The SAM program is discussed in Section 5.3.6. Based on data from Guy *et al*. (1966), Julien (1995) has subdivided the dominant mode of sediment transport into three zones. Using a ratio of shear velocity to fall velocity, and the ratio of depth

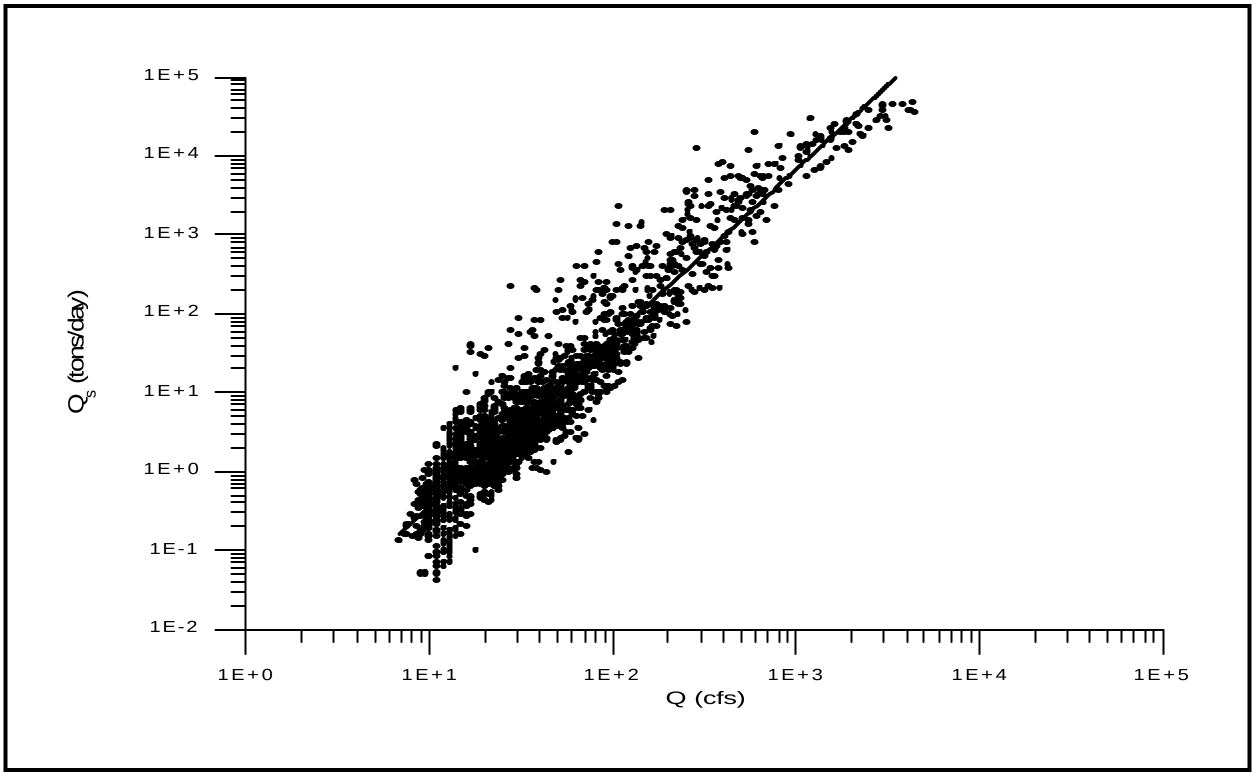


Figure 5.1 Fannegusha Creek Suspended Sediment Discharge

Table 5.4 Total Suspended Sediment Discharge Regression Relationships

Station No.	Station Name	a	b	R^2	N
7273100	Hotopha Creek near Batesville, MS	0.0115	1.828	0.82	2391
7274252	Otoucalofa Creek Canal near Water Valley, MS	0.0117	1.691	0.79	2648
7275530	Long Creek near Pope, MS	0.0029	2.016	0.89	2107
7277700	Hickahala Creek near Senatobia, MS	0.00184	2.031	0.82	2427
7277730	Senatobia Creek near Senatobia, MS	0.0052	1.956	0.83	964
7285400	Batupan Bogue at Grenada, MS	0.0025	1.832	0.89	2638
7287150	Abiaca Creek near Seven Pines, MS	0.0016	2.172	0.84	365
7287160	Abiaca Creek at Cruger, MS	0.0010	2.17	0.64	365
7287355	Fannegusha Creek near Howard, MS	0.0026	2.138	0.87	2156
7287404	Harland Creek near Howard, MS	0.0036	2.139	0.86	2122

Table 5.5 Bed Material Load (Sand Fraction) Relationships at Gaging Sites in the Yazoo Basin, Mississippi

Station No.	Station Name	a	b	R^2	N
7285400	Batupan Bogue at Grenada, MS	0.000000468	1.549	0.77	340
7287150	Abiaca Creek near Seven Pines, MS	0.00000362	1.506	0.62	138
7287160	Abiaca Creek near Cruger, MS	0.00000000395	2.37	0.73	108
7287355	Fannegusha Creek near Howard, MS	0.000000576	1.745	0.97	13
7273100	Hotopha Creek near Batesville, MS	0.000000141	2.026	0.77	145
7287404	Harland Creek near Howard, MS	0.000000152	1.596	0.81	531
7277700	Hickahala Creek near Senatobia, MS	0.0000000122	2.237	0.65	406
7275530	Long Creek near Pope, MS	0.000000599	1.692	0.86	214
7277730	Senatobia Creek near Senatobia, MS	0.000000679	1.645	0.85	10

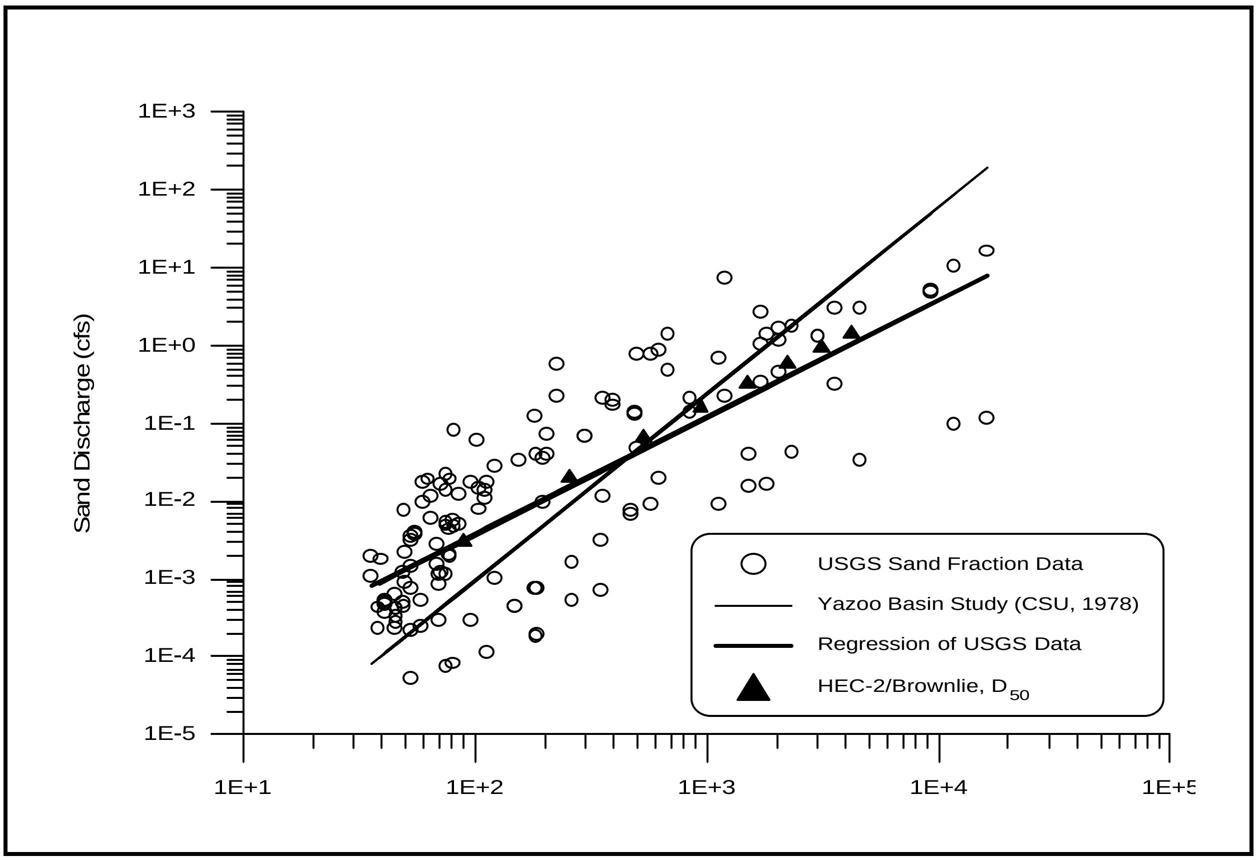


Figure 5.2 Comparison of Sediment Relationships for Abiaca Creek, Site No. 6

to particle size, the dominant sediment transport mode can be subdivided into bedload, mixed load, and suspended load.

Suspended load is dominant for values of the shear velocity to fall velocity ratio greater than 2.5. For a hypothetical range of particle sizes from 1 mm to 0.063 mm, slope from 0.0005 to 0.001, and depth from 0.015 m to 1.52 m, the range of values for the shear velocity to fall velocity ratio varies from a minimum of 2.6 to a maximum of 379. Therefore, for most conditions occurring within the DEC streams, the dominant transport mechanism is suspended load, which implies that the suspended sand discharge closely approximates the total bed material load. There is close agreement between the observed suspended sand discharge and the Brownlie procedure for total bed material load in sand channels. The dominance of suspended sand transport in the DEC streams is substantiated by the Guy *et al*. (1966) and Julien (1995) analysis; therefore, the suspended sediment discharge was used in this investigation as the total bed material load for sites at which the data are available. The Brownlie procedure will be used to compute total bed material load in the absence of measured data. Nash (1994) assumed that the bedload contribution is relatively insignificant as compared to total load. Others have also made this assumption (e.g., Benson and Thomas, 1966; Biedenharn *et al*., 1987; and Dunne, 1979).

5.1.5 WATERSHED HYDROLOGY

The hydrology of a river reach is defined by the recorded flow data, flow duration, bankfull discharge, and annual hydrograph shape. The mean annual flow of a river gives an indication of the size of a stream and is readily available from discharge records. Based on the mean annual flow, a stream is classified as being small if the mean annual discharge is less than 10,000 cfs (283 m^3/s) and the bankfull discharge is less than 50,000 cfs (1,416 m^3/s).

Bankfull discharge and effective discharge are extensively discussed in this manual (Section 5.3.1). The effective discharge should be computed and utilized in design calculations.

The shape of the annual hydrograph depends on the route in which runoff flows to a stream and the storage characteristics of the basin. The hydrograph can be considered an indicator of the sediment yield from the drainage basin and a reflection of the climate in which the stream reach is located. Rainfall intensity, number of precipitation events, and seasonal distribution of precipitation are factors that affect the sediment yield and are reflected in the hydrograph. Man-made structures such as channel diversions and reservoirs can have a serious effect on flow rate and storage capacity. These effects are reflected in the shape of the hydrograph and the skewness of the discharge. The land use in the region will also affect the shape of the hydrograph. While it is difficult to exactly quantify these effects from the hydrograph, the shape, nonetheless, can give a good insight into basin characteristics.

5.1.5.1 Project Hydrology Considerations

The hydrology of the watershed determines one of the fundamental driving forces of the system, i.e., the amount of water flowing across the watershed and through the stream system. Flowing water shapes the watershed and channel systems through erosion processes. The following section will present concepts of hydrology essential to evaluating runoff and sediment transport potentials.

5.1.5.1.1 *Gage Data*

Historical and real time stream gaging data are available for many gaged streams. These data along with a wide variety of useful information may be easily downloaded by those with Internet access at the following USGS web site: **http://waterdata.usgs.gov/**.

5.1.5.1.2 *Frequency Analysis*

The recurrence interval for discharges at the ungaged sites can be determined from USGS regionalized relationships using regionalized hydrology procedure, for example Landers and Wilson (1991) was used for the DEC Project. Because of the relatively short period of record for the gaged sites, less than ten years, this method was also utilized for the gaged sites. The relationship proposed by Landers and Wilson (1991) is:

$$Q_x = aDA^{b}S^{c}\,(L^{d}) \tag{5.1}$$

where Q_x = discharge at recurrence interval x years;
DA = drainage area in square miles;
S = channel slope, ft/ft;
L = major drainage path length; and
a, b, c, d = coefficients and exponents are listed in Table 5.6

Of course, other more computationally sophisticated hydrology models are available. Some of these are discussed in Section 5.1.5.2.

Table 5.6 Regional Discharge Recurrence Interval Coefficients and Exponents

Recurrence Interval	a	b	c	d
2 year	66.2	0.88	0.51	-0.11
5 year	94.7	0.93	0.51	-0.15
10 year	122	0.96	0.49	-0.19
25 year	164	0.99	0.47	-0.24
50 year	197	1.00	0.45	-0.26
100 year	230	1.00	0.44	-0.25

The 2-year recurrence interval was used as an index value by which watersheds could be compared, and by which discharges computed for one location within a watershed could be transferred to other locations within the same watershed.

5.1.5.1.3 *Flow Duration Curve*

A flow duration curve is a cumulative distribution function of discharges, as shown in Figure 5.3. A cumulative distribution diagram is prepared by dividing the discharge data into equal width classes. A count of the number of discharges in each class is made to make a histogram, and then adding each bar of a histogram to construct the cumulative distribution function.

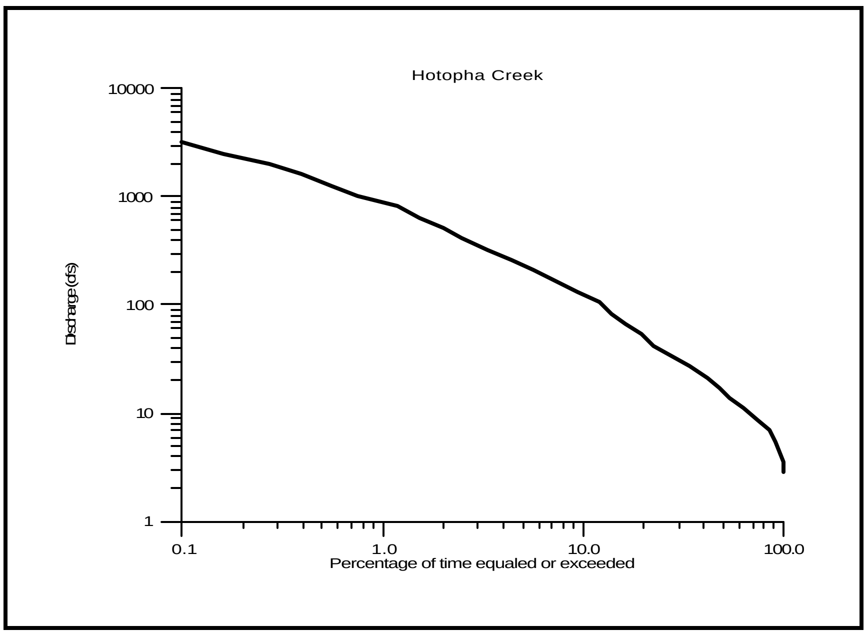


Figure 5.3 Cumulative Distribution Function of Discharge for Hotopha Creek, Mean Daily Data

The USGS flow duration procedure divides the data into 35 classes. The lowest class is zero, with a class width of 0 to 0. The next class width is 0 to the minimum discharge value. The remaining 33 classes are determined by subtracting the logarithm of the minimum discharge from the logarithm of the maximum value, and dividing by 33 to form equal logarithmic class widths. The upper interval must include the greatest measured discharge. After the class widths are set, a spreadsheet can be utilized to develop class counts for each year of the data and histogram values for equal classes can be directly added to develop the histogram for the total period of record. For example, histograms for 35 years of record may be developed in 5-year increments and can be added to form the total data set histogram. Equal width arithmetic classes can also be used to develop the flow duration relationship. Although these equal width classes can give better definition or the higher discharge values, representation or the low discharges will be masked by the relatively larger class intervals at the low discharge portion of the histogram. Arithmetic

class widths may give better definition in the effective discharge range, and poorer definition for computation of sediment yield, or for low flow water quality and habitat investigations.

Flow duration relationships were developed for ten USGS gaging stations within the DEC region for the mean daily record and for the 15-minute interval record. The results of these computations are presented in Figure 5.4 for the mean daily data, and Figure 5.5 for the 15-minute data.

5.1.5.1.4 *Watershed Data*

An area of land that drains to a single outlet or waterbody is called a watershed. Watershed boundaries follow the ridgelines and topographic divides that separate lands draining to different surface waters. Before a watershed plan can be created, whether for erosion control, water quality, or some other purpose, it is essential to know what exists in the watershed. The following sections briefly describe the fundamental information needed to characterize watershed hydrology and define boundary conditions for any subsequent hydraulic analyses.

5.1.5.1.5 *Watershed Boundaries and Areas*

A fundamental step in any watershed analysis is to delineate the boundary of the watershed above some point of interest and determine the contributing land area within that boundary. Watershed boundaries may be delineated by several means at various levels of accuracy. Watersheds may also be defined at many different scales and sizes from the scale of the Mississippi River watershed to the scale of the many thousands of streams and rivers that make up smaller watersheds within the larger Mississippi Basin. Perhaps the most common approach is to use USGS topographic maps at either 1:24,000 or 1:100,000 scales to identify the contributing area above the watershed outlet by tracing ridgelines determined from elevational contours. The technique for determining watershed boundary on a topographic map is to start at the base level and, working uphill, mark the ridgeline. The decision as to whether a particular piece of ground is "in" or "out" of the watershed may be determined by examining whether the area of interest flows to the stream above the base level or watershed outlet. Three simple rules help in this determination:

1. Water tends to flow perpendicularly across contour lines;
2. Ridges are indicated by contour "V"s pointing downhill; and
3. Drainages are indicated by contour "V"s pointing upstream.

Once the watershed boundary has been delineated, the area within may be estimated using a planimeter, tracing paper and a grid overlay, or by digitization using a digitizing tablet and an appropriate software package. More sophisticated approaches include the use of Geographic Information Systems (GIS) and digital elevation models to compute watershed areas. While it is recognized that most of the watershed is, in all likelihood, on a slope, the area that is reported is the horizontal projection of the watershed boundary. In regions where extensive drainage or supply networks are linked across former watershed divides, care must be taken to adequately represent

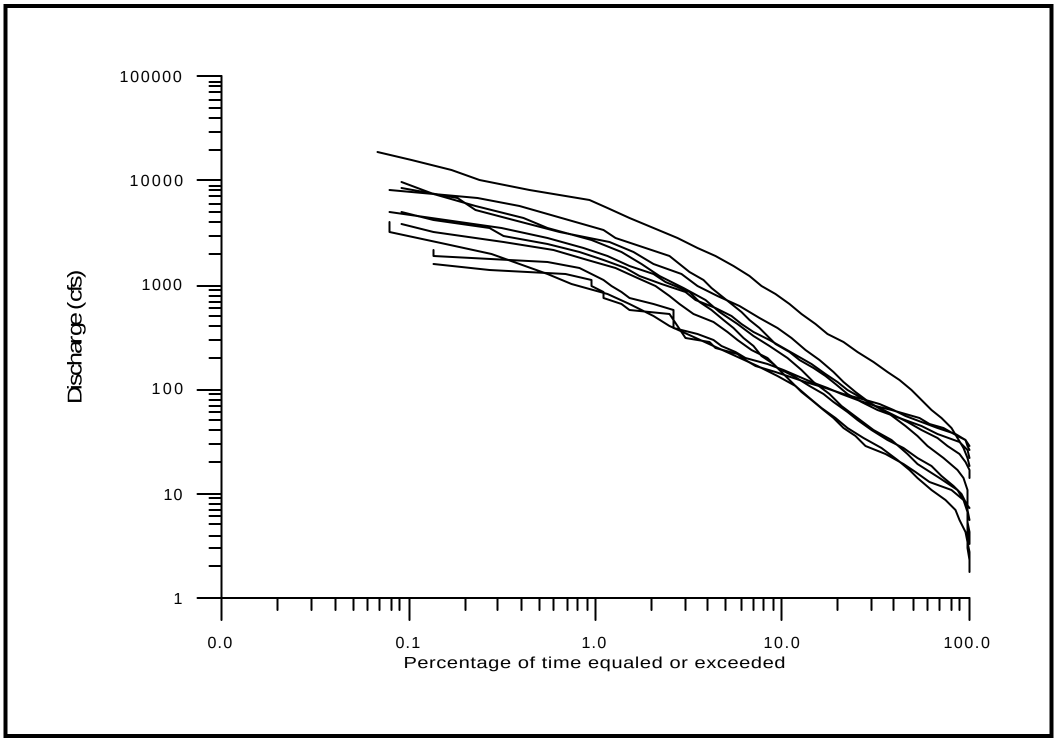


Figure 5.4 Flow Duration Relationships for Mean Daily Data on Ten Gages

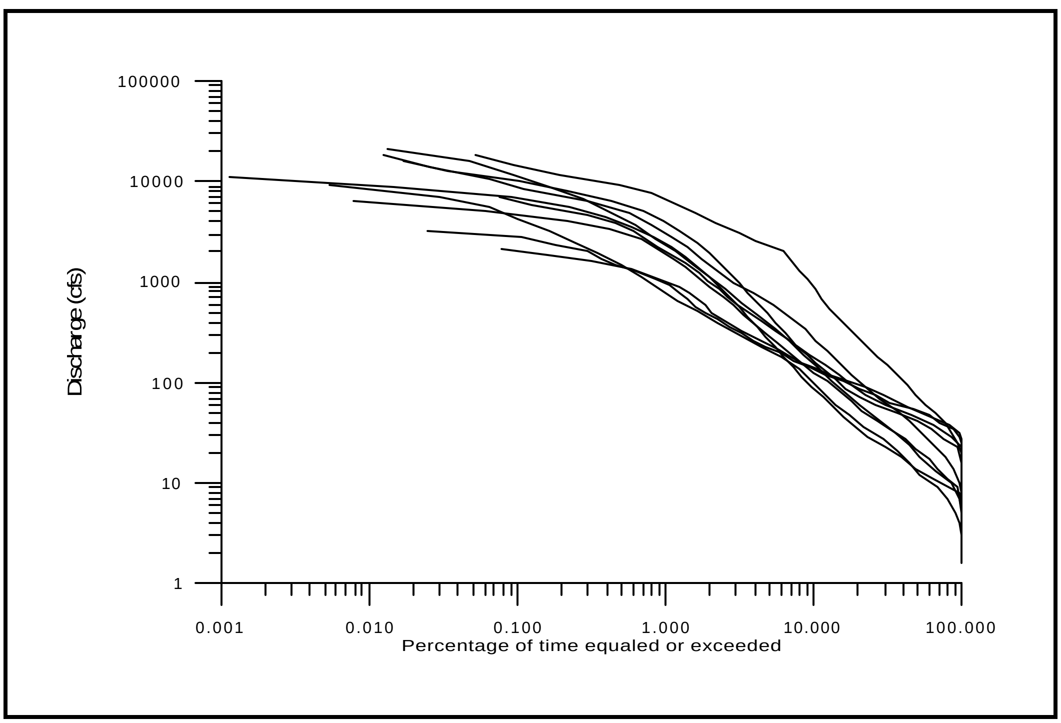


Figure 5.5 Flow Duration Relationships for 15-Minute Data

the area contributing water to the basin outlet. Field reconnaissance is an essential component of most watershed studies, particularly with regard to urban and agricultural drainage networks and the behavior of hydraulic structures.

Existing digital coverages of watershed boundaries sometimes preclude the need for watershed delineation. In most areas of the country, the boundaries of large watersheds (on the order of 1,000 square miles) have already been digitized by federal and/or state agencies. These boundaries are widely available at the scale of river basins and major subbasins but availability of small watershed boundaries varies from state to state. The USGS assigns a system of numerical codes that provides information on location, scale, and hierarchy/nesting for each basin. River basin scale watersheds with eight and eleven digit codes are widely available whereas fourteen digit "hydrologic units" are much less available. Fourteen digit watersheds, when available, are an excellent resource for watershed planning and management activities and usually depict watershed areas on the order of 10-100 mi^2. It has been estimated that over 93 percent of streams nationwide have drainage areas of less than 23 square miles (Leopold *et al*., 1964). Planning on the scale of these small watersheds, therefore, has the potential to positively effect a great majority of surface waters.

5.1.5.1.6 *Watershed Attributes – Geographic Information Systems*

Once a watershed has been identified, a number of parameters can be calculated that aid in describing and quantifying the characteristics of the watershed. The determination of several watershed parameters provides information that is useful in making decisions about how to manage the watershed in addition to simply describing it. Organizations in the United States that offer GIS resources include:

- Alaska Geospatial Data Clearinghouse
- Arkansas, Center For Advanced Spatial Technologies
- Arizona Geographic Information Council
- California
    - Teale Data Center
    - California Geographic Information Council
- Colorado Geographic Information Coordinating Committee
- Florida Data Directory
- Idaho Geographic Information Center
- Illinois Natural History Survey
- Iowa
    - Iowa Geographic Information Council
    - Iowa Geospatial Data Clearinghouse
    - Iowa Dept. of Natural Resources
- Kansas
    - Kansas Data Access and Support Center (DASC)
    - Kansas Geographic Information Systems Policy Board
- Kentucky Geographic Information Advisory Council
- Louisiana Geographic Information System Council

- Maine Office of GIS
- Maryland State Government Geographic Coordinating Committee
- Massachusetts Executive Office of Environmental Affairs
- Michigan Resource Information System (MIRIS)
- Governor's Council on Geographic Information
- Minnesota Land Management Information Center
  - MN GIS/LIS Consortium
  - MN DNR MIS Bureau
- Missouri Spatial Data Information Service
- Montana
  - Montana State Library GIS Program
  - Montana Local Government GIS Coalition
- Nebraska
  - Nebraska Geospatial Data Clearinghouse
  - Nebraska Natural Resources Commission
- Nevada Bureau of Mines and Geology
- New Hampshire Resource Net
- New Jersey GIS Resource Guide
- New Mexico GIS Advisory Council
- New York State Center for Technology in Government
- North Carolina Center for Geographic Information and Analysis
- North Dakota State Data Center
- Ohio Geographically Referenced Information Program
- Oklahoma Spatial and Environmental Information Clearinghouse (SEIC)
- Oregon's Center for Geographic Information Systems
- Pennsylvania Mapping And Geographic Information Consortium
- Rhode Island Geographic Information System Data
- Texas GIS Planning Council
- Utah Automated Geographic Reference Center
- Vermont
  - Vermont Geographic Information System
  - ANR-ISP Project
- Washington State Geographic Information Council
- West Virginia Library Commission
- Wisconsin
  - Wisconsin Land Information Clearinghouse
  - Wisconsin Land Information Clearinghouse NSDI node
- Wyoming Geographic Information Advisory Council
- U.S. Geological Survey (USGS)
  - Global Land Information System
  - Index to USGS Digital Data product availability
  - USGS Explanation of GIS
- Federal Geographic Data Committee

- Environmental Protection Agency
- U.S. Census Bureau
  - TIGER files
  - TIGER Map Browser
- U.S. Fish and Wildlife Service
  - National Wetlands Inventory
- NASA
- U.S. Dept. of Agriculture (USDA) - Natural Resources Conservation Service

5.1.5.1.7 ***Land Use***

Land use may have a profound effect on hydrologic processes including runoff quantity, quality, and timing, infiltration, and sediment transport and delivery. For this reason, the effects of land use must be accounted for in any comprehensive watershed analysis. In many watersheds, land use changes have directly resulted in accelerated geomorphic activity and excessive sedimentation. The effect of land use alterations may be generalized as a change in the natural storage in a watershed. This alteration may lead to an increase in both runoff volume and rate with an associated increase in erosion and sedimentation potential. Even where land use changes do not result in a significant increase in upslope erosion, altered runoff delivery may increase channel erosion and downstream sediment deposition. Changes in the magnitude, relative proportions, and timing of sediment and water delivery result in loss of water quality via a wide variety of mechanisms. These mechanisms include changes in channel bed material, increased suspended sediment loads, loss of riparian habitat due to stream bank erosion, and changes in the predictability and variability of flow and sediment transport characteristics relative to aquatic life cycles (Waters, 1995).

There is an increasing variety of sources of land use / land cover data. The most common forms are aerial photography and digital satellite imagery. Important considerations in selecting land use / land cover information include scale, resolution, and applicability to current conditions (vintage). As an example, consider Landsat Thematic Mapper satellite imagery (30 m pixels) from this decade that is available for many parts of the country. These data are usually classified into several types of land cover including various hardwood and evergreen forest types, agricultural land cover types, and differing densities of urban and suburban development. For practical purposes, especially in predominately forested areas, Landsat data may be used to differentiate evergreen from deciduous forest, grassland from most cropland, and to identify urban centers without substantial forest canopy cover. Attempts to "push" the data beyond these limits will often result in erroneous results. It is also important to remember that remotely-sensed land use / land cover information may not necessarily be representative of below-canopy processes. For example, forested residential areas or severe disturbance to streambanks and riparian areas may be indistinguishable from relatively pristine locations if covered by a forest canopy. Furthermore, changes in land use that may have occurred since the imagery was taken will not be reflected in the data. Pixel size and cell averaging techniques also limit data applicability in identifying features such as narrow riparian zones. Field reconnaissance and recognition of the limitations of remotely sensed data through the examination of error analyses and metadata are critical in estimating model parameters and determining appropriate action.

Sources of aerial photography include the geographic information clearinghouses described above as well as local or state government planning agencies, forest products companies, and federal agencies such as the U.S. Army Corps of Engineers, Natural Resources Conservation Service (formerly SCS), Farm Services Agency, U.S. Forest Service, Bureau of Land Management, etc. If the desired land use / land cover data are not available in an appropriate digital form, the techniques described for determining watershed area may be used in conjunction with maps or photos to determine the relative proportions of various land uses within the watershed boundary.

From a practical engineering hydrology standpoint, representative land use information is essential to determining appropriate parameters to describe runoff-infiltration processes. The most common forms of such parameters are the SCS Curve Number (CN) and the Rational "C" runoff coefficient. If a lumped parameter model such as HEC-1 is used, an area weighted curve number or runoff coefficient is usually computed for the entire watershed or, if the watershed has been discretized into sub-watersheds, each sub-watershed. If a distributed-parameter hydrologic model is used to simulate watershed hydrology, a parameter representing runoff-infiltration processes associated with land use and soil type will be required for each analysis cell. Export coefficients or Universal Soil Loss Equation parameters associated with sediment delivery from different land covers / uses and conservation practices also provide critical information in understanding and predicting watershed response.

5.1.5.1.8 *Soils*

Soil physical properties are also important determinants of runoff-infiltration processes. Soil pore space is a giant reservoir that provides the primary buffering of precipitation delivered irregularly to the surface of the Earth. As with land use, some knowledge of soil properties in the watershed of interest is essential to the selection of appropriate model parameters. Even in the simplest hydrologic models such as the SCS CN approach, soil physical properties are commonly used in conjunction with land use / land cover information to select parameters describing runoff-infiltration processes.

Physical, chemical, and biological properties of various soil mapping units are readily available in the form of Natural Resources Conservation Service (NRCS) County Soil Surveys. Depending on the level of detail required in a particular hydrologic analysis, a few or all of the following data provided by NRCS may prove useful:

Soil maps (usually 1:24,000 scale)
Soil texture of each mapping unit
Soil permeability (usually overestimated in county soil surveys, particularly for non-agricultural soils)
Soil Hydrologic Groups (A, B, C, D)

As described above, digital soil layers (DSLs) are available for many parts of the country. Although DSLs may only contain the soil mapping unit name in addition to the basic GIS descriptors, each polygon or cell in a DSL may be linked to additional "lookup tables" so that the full range of soil physical properties described in a county soil survey may be utilized in hydrologic analyses.

5.1.5.1.9 *Weather and Climatological Data*

An enormous variety of weather and climatological data are available from the National Oceanic and Atmospheric Administration's (NOAA) National Climatic Data Center (NCDC) on the world wide web at **http://www.ncdc.noaa.gov/**. NCDC is the world's largest active archive of weather data. In addition to producing numerous climate publications and responding to data requests from all over the world, NCDC supports a three tier national climate services support program that includes NCDC, Regional Climate Centers (RCC's), and State Climatologists. Weather and climatological data are useful in identifying the magnitude and rate of precipitation associated with a particular design storm, temperature, humidity and other data used in modeling evapotranspiration processes, and long-term records of rainfall for locations of interest.

5.1.5.1.10 *Watershed Climate and Hydrology*

For many years, the USGS has been involved in the development of regional regression equations for estimating flood magnitude and frequency at ungaged sites. These regression equations are used to transfer flood characteristics from gaged to ungaged sites through the use of watershed and climatic characteristics as explanatory or predictor variables. The USGS-developed regression equations are generally unbiased, reproducible, and easy to apply. The standard errors of estimate or prediction generally range from 30-60 percent, with 21 states having standard errors in this range. There are fourteen states where there is at least one hydrologic region within the state with a standard error less than 30 percent. The largest standard errors are for equations applying the western portion of the nation where the at-site variability of the flood records is greater, where the network of unregulated gaging stations is less dense and there are more difficulties in regionalizing flood characteristics, and the flood records are generally shorter. The smallest standard errors are generally for equations developed for the eastern portion of the country where the converse of the above conditions is generally true. Regionalized discharge is discussed in Section 3.1.2.

In geomorphic or hydraulic analyses of fluvial systems, it is often necessary to identify a "channel-forming" or "effective" discharge that exerts the most morphologic influence over channel geometry. The smaller of the bankfull or estimated two year discharge (Q2) is frequently used as surrogate value of effective discharge when flow and sediment monitoring data are insufficient to directly compute the actual effective discharge. Estimates of Q2 derived from the above equation may be particularly useful in assessment and design under these common circumstances. Since hydrology determines the boundary conditions for hydraulic analyses, the accuracy of any subsequent hydraulic analyses will only be as good as the hydrologic information upon which they are based.

5.1.5.2 Computation of Project Hydrology - Hydrology Models

Although there are many different computer models that can be utilized in developing project hydrology, only HEC-1 (USACE, 1985) and CAS2D (Julien *et al*., 1995) will be discussed. HEC-1 is probably the hydrologic model most familiar to USACE personnel, and CAS2D is the one of the surface-water hydrologic models selected to be included in the watershed modeling system under development by the USACE.

5.1.5.2.1 *HEC-1*

Ponce (1985) describes HEC-1, subtitled *Flood Hydrograph Package*, as a program designed to be used for the simulation of flood events in watersheds and river basins (USACE, 1985). The river basin is represented as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin referred to as subbasin. Component description requires the knowledge of a set of parameters and mathematical relationships describing the physical processes. The result of the modeling is the computation of streamflow hydrographs at desired locations within the river basin.

A river basin is represented as an interconnected group of subbasins. Within each subbasin, the hydrologic processes are represented by average parameter values. For hydrologically nonhomogeneous subbasins, further subdivision may be necessary to ensure that average parameter values are representative of each subbasin entity.

HEC-1 is an event model; therefore, it has no provision for soil moisture recovery during periods of no precipitation, with simulations being limited to a single-storm event. The model calculates discharges only, although stages can be indirectly determined through ratings supplied by the user. Alternatively, the results of HEC-1 can be used as input to HEC-2, which calculates stages based on discharge by using steady gradually varied flow principles. In HEC-1, stream channel routing is accomplished by hydrologic methods. Therefore, the model does not account for the dynamic effects that are present in rivers of mild slope. Reservoir routing is based on the modified Puls technique, which may not be applicable in cases where reservoirs are operated with controlled outflow.

HEC-1 model components simulate the rainfall-runoff process as it occurs in a river basin. Mathematical relations are intended to represent individual meteorological, hydrologic, and hydraulic processes encompassing the rainfall-runoff phenomena. The processes considered in HEC-1 are (1) precipitation, (2) interception/infiltration, (3) transformation of effective precipitation into subbasin runoff, (4) addition of baseflow, and (5) flood hydrograph routing, either in stream channels or reservoirs.

Rough estimates of HEC-1 model parameters can usually be obtained from individual experience or by other empirical means. Calibration using measured data, however, is the preferred way of estimating model parameters. With rainfall-runoff data from gaged catchments, the mathematical optimization algorithm included in HEC-1 can be used to estimate some model parameters. Using regional analysis,

parameters obtained in this way can be transferred to ungaged catchments of similar hydrologic characteristics (USACE, 1985).

Information on the availability of HEC-1 can be obtained from the U.S. Army Corps of Engineers Hydrologic Engineering Center internet site, **http://www.waterengr.com/ hecprog2.htm.**

5.1.5.2.2 *CASC2D*

Ogden (1998) describes CASC2D as a fully-unsteady, physically-based, distributed-parameter, raster (square-grid), two-dimensional, infiltration-excess (Hortonian) hydrologic model for simulating the hydrologic response of watersheds subject to an input rainfall field. Major components of the model include: continuous soil-moisture accounting, rainfall interception, infiltration, surface and channel runoff routing, soil erosion and sediment transport. CASC2D development was initiated in 1989 at the U.S. Army Research Office (ARO) funded Center for Excellence in Geosciences at Colorado State University. The original version of CASC2D has been significantly enhanced under funding from ARO and the U.S. Army Corps of Engineers Waterways Experiment Station (USACEWES). CASC2D has been selected by USACEWES as its premier two-dimensional surface water hydrologic model, and is one of the surface-water hydrologic models support by the Watershed Modeling System (WMS) under development at Brigham Young University.

CASC2D is a state-of-the-art hydrologic model that takes advantage of recent advances in Geographic Information Systems (GIS), remote sensing, and low-cost computational power. Compared with the USACE standard practice surface water hydrology model HEC-1, CASC2D offers significant improvements in capability. HEC-1 requires the division of study watersheds into sub-catchments that are assumed to be hydrologically uniform, while CASC2D allows the user to select a grid size that appropriately describes the spatial variability in all watershed characteristics. Furthermore, CASC2D is physically-based; CASC2D solves the equations of conservation of mass and energy to determine the timing and path of runoff in the watershed. More traditional approaches such as HEC-1 rely on more conceptualizations of runoff production. The physically-based approach is superior when the modeler is interested in runoff process details at small scales within the watershed. Physically-based hydrologic models are also superior when trying to predict the behavior of ungaged watersheds where calibration data do not exist.

The following paragraphs describe CASC2D input requirements and simulation capabilities. These descriptions are intended for general informational purposes. For more detailed descriptions, see Julien *et al*. (1995) and Ogden (1997).

An explicit, two-dimensional, finite-difference, diffusive-wave scheme is used to route overland flow in CASC2D (Julien *et al*., 1995). The Manning equation is used to calculate overland flow velocities, requiring input of a map of spatially-varied Manning roughness coefficient values for overland flow. The routing scheme is shown conceptually in Figure 5.6.

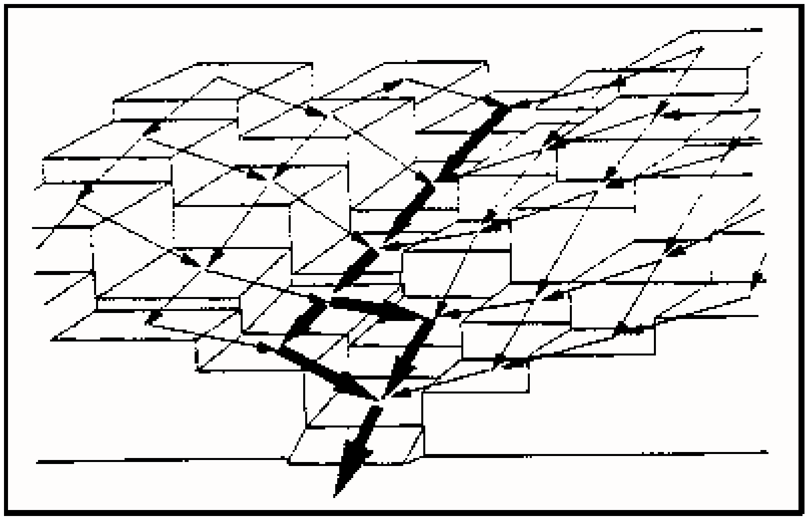


Figure 5.6 Conceptual Sketch of CASC2D Overland Flow Routing (from Ogden, 1998)

The grid elevations shown in Figure 5.6 represent water surface elevations, rather than land-surface elevations because of the diffusive wave formulation.

Overland soil erosion is calculated in each CASC2D model grid using the Kilinc (1972) approach as implemented by Johnson (1997). This method calculates the sediment discharge q_s in tons per second per meter width of overland flow plane using:

$$q_s = 25500\ q_o^{2.035}\ S_o^{1.664} \tag{5.2}$$

where: q_o = overland flow discharge per unit width (m^2/s) in x and y directions, respectively; and
S_o = land-surface slope in x and y directions, respectively.

The Kilinc (1972) method is empirical and based on plot-scale data collection using a bare, sandy-soil. In CASC2D, three different size-classes of material are considered; sand, silt, and clay. Sediment transport is calculated using Eq. (5.2), and the net erosion/deposition of each size class is assumed proportional to the percentage of soil in each size class. The amount of sediment transport predicted using Eq. (5.2) is multiplied by an erosivity factor, and a land-use management factor to consider both of these effects. Both of these factors have values between 0 and 1.

Once overland sediment reaches the CASC2D channel network, silt and clay size fractions are routed as suspended or "wash" load. The sand size fraction is assumed to be deposited on the bed of the channel and routed as bed-load using Yang's (1973) method. Channel bed elevations are allowed to erode and aggrade in accordance with the rates of sediment influx and outflow.

CASC2D has been employed to simulate a number of study-watersheds with considerable success. Like any other hydrologic model, CASC2D is founded on assumptions of the relative importance of different hydrologic processes. Recent experiences with CASC2D have shown that in regions of infiltration-excess (Hortonian) runoff production, CASC2D is quite accurate at predicting runoff, even at internal locations within the watershed (Johnson *et al.*, 1993; Ogden *et al.*, 1998). The continuous simulation capability of CASC2D has been found to be particularly good for reducing the uncertainty in estimating initial soil-moisture conditions, and for improving calibration uniqueness (Ogden and Senarath, 1997; Ogden *et al.*, 1998). CASC2D has also proven to be valuable for studying extreme runoff events. The model was recently applied to study the extreme flood on the Rapidan River, Virginia, on June 27, 1995 (Smith *et al*. 1996), for the purpose of examining geomorphological changes; and the extreme urban flood event in Trenton, New Jersey (Stock, 1977) for the purpose of recommending stormwater management improvements. CASC2D is currently being applied to evaluate the impact of radar-rainfall estimation errors in a study funded by ARO, and in an NSF-sponsored study of the devastating flood that severely impacted Fort Collins, Colorado, on June 28, 1997.

The overland erosion/sediment transport capabilities of CASC2D were evaluated in detail by Johnson (1997). In upland areas, the method was shown to calculate sediment yield well with the acceptable range of -50% to +200%. Compared with actual field observations of annual sediment yield, CASC2D predictions were generally within 20% of observed values.

Further information on CASC2D capabilities and availability can be obtained from the U.S. Army Engineer Waterways Experiment Station web site, **http://chl.wes.army.mil /software/**.

5.1.6 METHODS FOR ASSESSING HISTORICAL RIVER STABILITY

The channel stability assessment phase of the systems approach requires developing an understanding of the total system dynamics. This understanding will allow the investigator to discriminate between channel reaches that are degradational, aggradational, or in a state of equilibrium, and also to catagorize channel banks as stable or unstable. A geomorphic investigation of the entire watershed is beneficial, and the detail of the geomorphic investigation depends on the level of effort required for the systems analysis. Through the geomorphic study, system responses, past and present, are determined, and all pertinent data are assimilated to form a picture of the overall system stability.

Various tools that facilitate the stability assessment are developed from the gathered information. The following sections present four typical tools used to assess channel stability. These include specific gage analysis, comparative thalweg analysis, analysis of cross section geometry, and aerial photography. A detailed field investigation is also extremely important in assessing channel stability because the physical characteristics of the stream are indicators of the dominant geomorphic processes occurring in the basin. Section 5.2 describes recommended procedures and methods for field investigations.

5.1.6.1 Specific Gage Analysis

Perhaps one of the most useful tools available to the river engineer or geomorphologist for assessing the historical stability of a river system is the specific gage record. According to Blench (1966):

> *There is no single sufficient test whether a channel is in-regime. However, for rivers, the most powerful single test is to plot curves of "specific gage" against time; if the curves neither rise nor fall consistently the channel is in-regime in the vicinity of the gaging site for most practical purposes.*

A specific gage record is simply a graph of stage for a specific discharge at a particular gaging location plotted against time. A channel is considered to be in equilibrium if the specific gage record shows no consistent increasing or decreasing trends over time, while an increasing or decreasing trend is indicative of an aggradational or degradational condition, respectively (Figure 5.7).

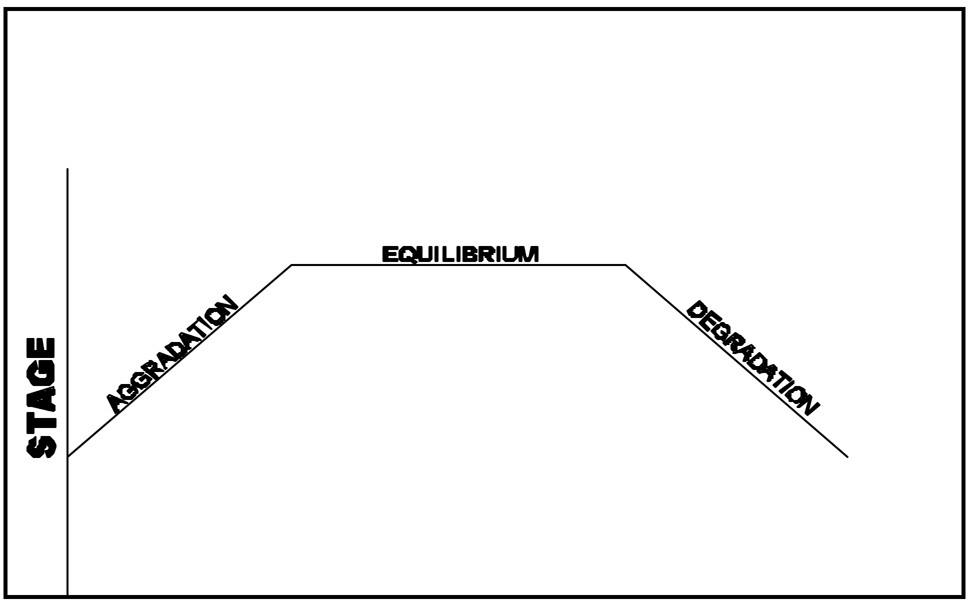


Figure 5.7 Definition Sketch of Specific Gage Record

Figure 5.8 is a hypothetical curve to help illustrate the procedure for developing a specific gage record. The first step in the development of a specific gage record is to establish the stage-discharge relationship at the gage for the period of record being analyzed. The stage-discharge relationship is generally depicted in the form of a stage-discharge rating curve which is a plot of the measured water discharge versus the observed stage at the time of measurement. A rating curve is developed for each year in the period of record. A regression curve is then fit to the data and plotted on the scatter plot. The regression curve is often fit by "eye," but the use of a curve fitting technique is recommended in order to provide a more consistent procedure that minimizes subjectivity. Since the specific gage record reflects only observed data it is important that the regression line does not extend beyond the limits of the measured

data. For this reason there may be some years where the gage reading for very large or small discharges may have to be omitted. In this case there will simply be a gap in the specific gage record for that year.

In some instances, there may be insufficient data to construct a rating curve for each year in the period of record. In these situations it may be necessary to combine the data from several years in order to obtain a large enough data set to develop a reliable rating curve. In this case the rating curve would reflect conditions over some time periods such as five or ten years.

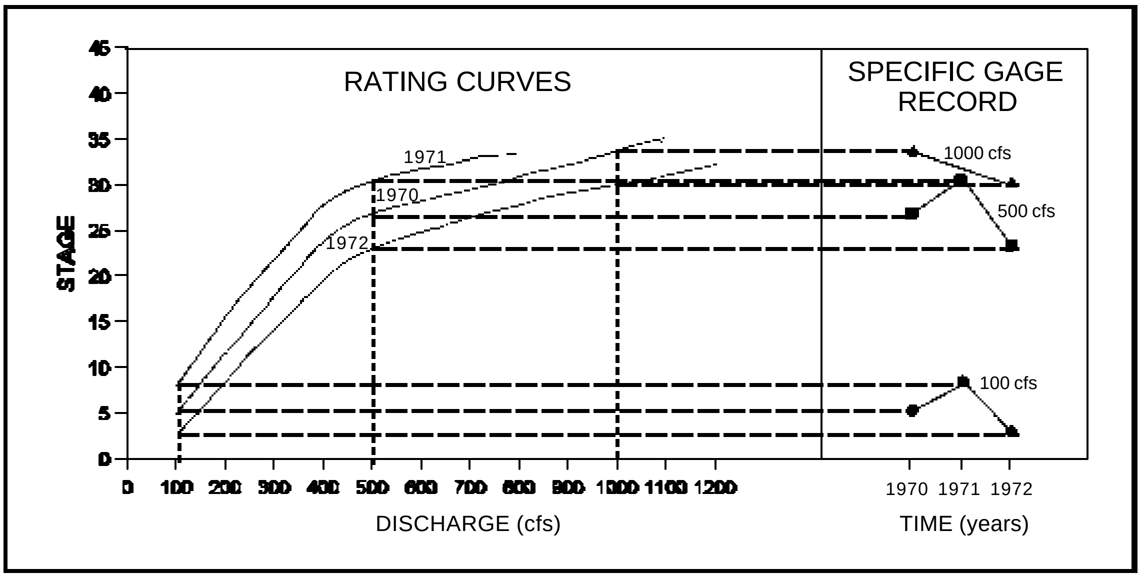


Figure 5.8 Development of Specific Gage Record

Once the rating curves have been developed, the discharges to be used in the specific gage record must be selected. This selection will depend, in a large part, on the objectives of the study. It is usually advisable to select discharges that encompasses the entire range of observed flows. This is important because the behavior of the low and high flows are often quite different. For this example discharges of 100 cfs, 500 cfs, and 1,000 cfs were selected. The stage for each of these flows at each time period is determined from the regression curves in Figure 5.8. For example, the stage for 500 cfs was 26 feet in 1970, 29.8 feet in 1971, and 22.5 feet in 1972. This data is then plotted to produce the specific gage record shown in Figure 5.8. Note that in 1971 there is no observed point on the specific gage record for the 1,000 cfs flow because the rating curve only extended up to 800 cfs that year.

The development of a specific gage record is a simple, straightforward procedure. However, the interpretation of specific gage records is more complex. The following paragraphs provide examples of the use of specific gage records.

To utilize a specific gage record properly it is necessary to understand exactly what it is a specific gage depicts. Specific gage records are often used to show aggradational and degradational trends in a river. Aggradation and degradation are terms that are generally associated with the increase or decrease

in the bed elevation of a stream. Therefore, specific gage records are associated with the changes in the bed elevation. However, a specific gage record charts the change in the stage of the water surface for a given discharge through time, and does not necessarily reflect the behavior of the bed of the river. While it is true that in many cases, the lowering or raising of stages is a result of changes in the bed elevation, there are other factors other than the bed elevation that can affect the water surface stage. For this reason, one must be careful when assuming that the specific gage records reflects the behavior of the stream bed.

Specific gage records are often used to illustrate the response of a river to various alterations in the channel or watershed. Inglis (1949) used specific gage records to document the response of the Indus River downstream of the Lloyd Barrage (Figure 5.9). Inglis used a slight variation of the procedure described by defining the specific gage record based on rising and falling stages. According to Blench (1966):

> *The object of these curves was to show the relatively sudden regime changes due to barrage construction and later extension, each followed by a relatively slow trend (or secular change of regime) towards a new steady regime at a higher elevation than originally, due to sediment exclusion from the canals and reduced river flow.*

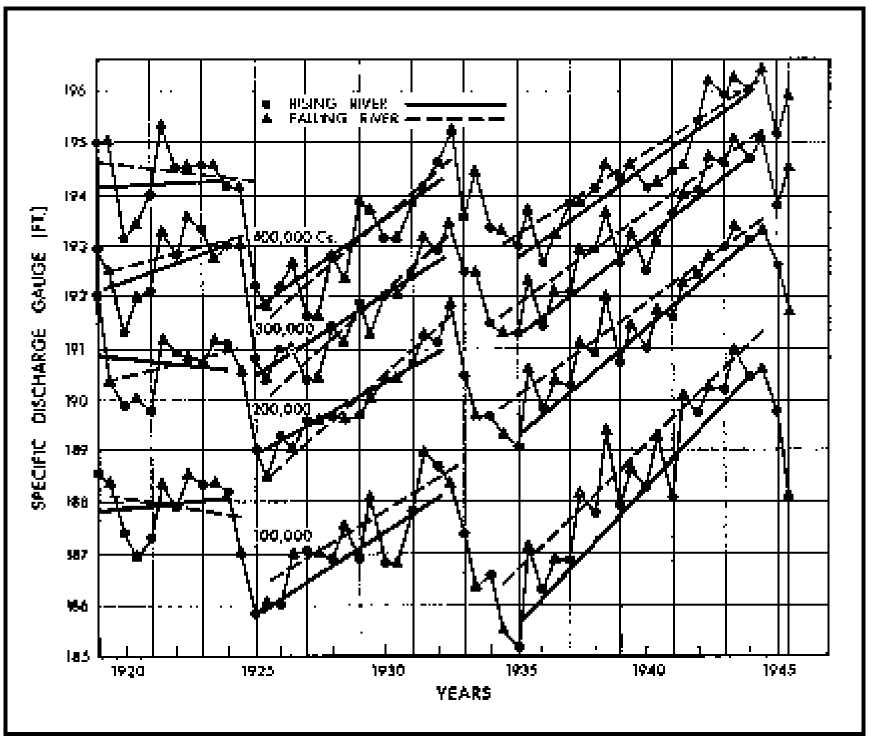


Figure 5.9 Specific Gage Record, Indus River Downstream of Sukkar Barrage (after Inglis, 1949)

Another example of the response of a river to the construction of a dam is illustrated in Figure 5.10 which shows the specific gage curves below the Trimmu Barrage, India (Galay, 1983). The specific gage

records clearly indicate the immediate lowering in water levels after the barrage was built. Figure 5.10 also indicates that the stages began a slow increase following the initial drop. Thus, the specific gage record provides a clear representation of the historical long-term channel response on this system.

On many river systems the response of the channel at low and high flows is entirely different. For this reason, it is often advisable to develop a specific gage record that covers the entire range of flows that the river encounters. The following example is presented to illustrate how the specific gage record can document the difference between the low flow and high flow response. Biedenharn (1983) used specific gage records to document the channel response of the Little Tallahatchie River below Sardis Dam in north Mississippi. Figure 5.11 shows the specific gage record for the Little Tallahatchie River at Belmont Bridge which is located approximately six miles below Sardis Dam in north Mississippi. The specific gage record was developed for the entire range of flows encountered on the river, ranging from a low flow of 500 cfs to a near bankfull flow of 5,000 cfs. As shown in Figure 5.11, the constant level of the specific gage record prior to 1939 indicates that both the low and high flows were fairly stable. With the closure of Sardis Dam in 1939 and the construction of five cutoffs immediately below the dam in 1941, the high flow stages showed an immediate and dramatic lowering. Between 1943 and 1950, the high flow stage began to increase as the channel began to aggrade. After about 1950, the stages appeared to have stabilized somewhat until about the late 1960's when the stages began to increase again. The response of the low flow stage was entirely different. Following the closure of Sardis Dam and the construction of the cutoffs, the low flow stage do not show any change. In fact, the low flow stages do not show any significant change until the late 1960s when they began to increase dramatically. Thus the most significant changes in the low and high water stages occurred during two different time periods and in different directions: the high flow stages lowered about 3 to 4 feet between 1939 and 1943 while the low flow stages rose about 3 feet during the period 1967 to 1980.

By definition, a specific gage record represents the variation in stage for a given discharge over time at a specific location on the river. Therefore, a specific gage record provides a picture of the river behavior at one point on the river, and does not necessarily reflect how the river is behaving upstream or downstream of that location. For this reason it is helpful if specific gage records can be developed at various locations along the river in order to illustrate how the overall system has responded. This often allows the engineer to develop an understanding of the connectivity of the system. This point is illustrated in Figure 5.12 which shows the specific gage records for the Red River at Shreveport, Louisiana and Alexandria, Louisiana for a discharge of 100,000 cfs for the time period 1895 to 1985. A discharge of 100,000 cfs is a fairly high flow with a return period just under 2 years. Shreveport is located approximately 160 miles upstream of Alexandria. Throughout the 1800's, the Red River was blocked by a huge log jam which extended about 80 miles upstream and downstream of Shreveport. This log jam, which was known as the Red River Raft, was finally removed in the late 1800's. With the removal of this blockage, the stages at Shreveport were lowered dramatically. According to the specific gage record (Figure 5.12), the stages were lowered approximately 15 feet by 1940. However, during this same time period (1895 to 1940), the stage at Alexandria actually increased approximately 4 feet (Figure 5.12), possibly as a result of the increased

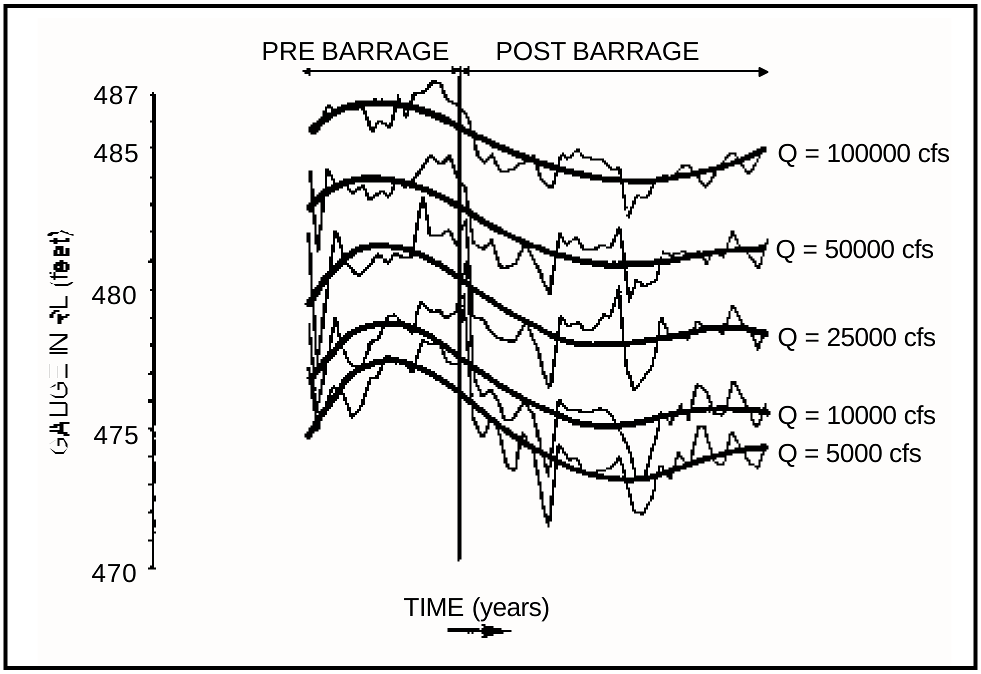


Figure 5.10 Specific Gage Record Below Trimmu Barrage, India (after Galay, 1983)

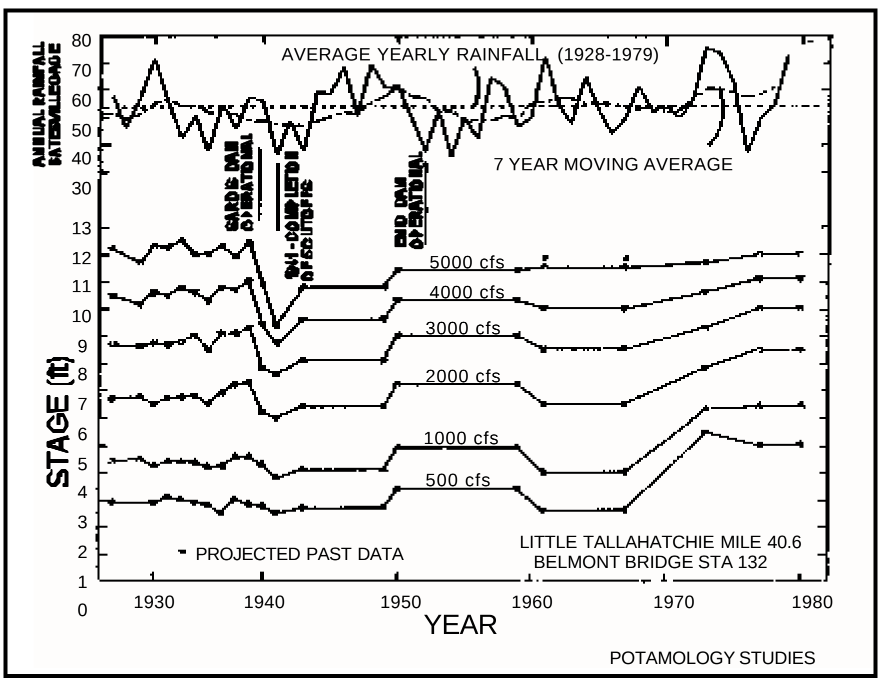


Figure 5.11 Specific Gage Record, Little Tallahatchie River Below Sardis Dam, Mississippi (after Biedenharn, 1983)

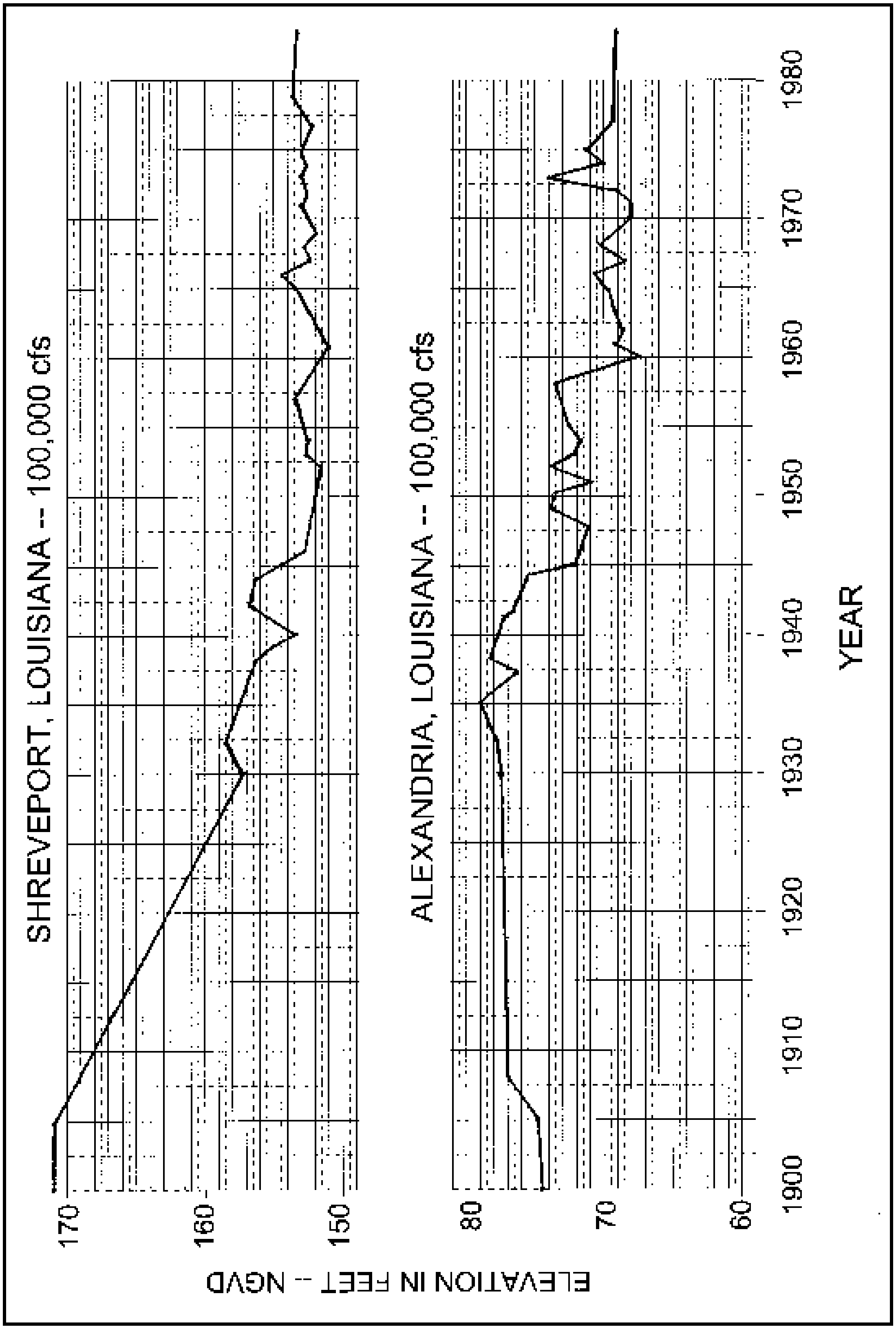


Figure 5.12 Specific Gage Record on Red River at Shreveport, Louisiana and Alexandria, Louisiana

sediment load from the degradational reach upstream. Thus, the specific gage records indicate that two entirely different modes of channel response were occurring: degradation at Shreveport and aggradation at Alexandria.

An excellent example of the use of specific gage records to illustrate the varied response along a river system is shown in Figure 5.13. Figure 5.13 shows specific gage records for seven gaging stations along the Mississippi River from Columbus, Kentucky to Red River Landing, Louisiana for near bankfull conditions for the time period 1860 to 1975 (Winkley, 1977). During the period 1933 to 1942, 16 manmade cutoffs were constructed on the river from just below Natchez to just above Helena. These cutoffs shortened the river approximately 160 miles. The immediate effect of these cutoffs is clearly shown (Figure 5.13) at the Natchez, Vicksburg, Arkansas City, and Helena gages where stages were lowered considerably. The most dramatic lowering occurred at Vicksburg and Arkansas City where stages were lowered approximately 15 and 12 feet, respectively. Since about 1950, the Natchez and Vicksburg gages have reversed their degradational trend and appear to be aggradational, while the stage at Arkansas City appears to have stabilized. In the upper reaches, Helena and Memphis are continuing on a downward trend. Further upstream, at the Columbus gage there has been no observed lowering that could be associated with the cutoffs. Thus the specific gage records provide a record of the complex response of the channel and a means of assessing the relative stability between various locations along the river. One of the most common mistakes in the utilization of specific gage records is to place too much emphasis on a short time period. The specific gage records on most rivers exhibit considerable variation about a mean value. There may even be cyclic patterns in the record. Therefore, localized trends in the specific gage record over relatively short time periods may not reflect a true long-term progression of the river. This is illustrated by examining the specific gage record for the Mississippi River at Arkansas City for the time period 1940 to 1974 (Figure 5.14). Looking at Figure 5.14, one can see how an engineer in 1974 might be tempted to conclude that degradational trend that had existed since 1940 had ended about 1967, and that the channel was now in an aggradational mode. The engineer might then use this aggradational assumption as the basis for design of channel improvement features such as levees, revetments dikes, or for making projections about long-term dredging quantities and channel response. However, when the period of record is extended to 1990 (Figure 5.15) it becomes apparent that the 1967 - 1974 period was just a short-term phenomenon, and that the channel is still on a degradational trend. Thus the design assumption mentioned above would have been in error.

Specific gage records are an excellent tool for assessing the historical stability at a specific location. However, specific gage records have two limitations. First, a specific gage record only indicates the conditions at a particular gaging station and does not necessarily reflect river response upstream or downstream of the gage. Second, a specific gage record does not provide any indication about future degradation or aggradation trends. Extrapolation of specific gage records into the future is extremely risky and is generally not recommended. Therefore, even though the specific gage record is one of the most valuable tools used by river engineers, it must be coupled with other assessment techniques such as slope analysis in order to assess reach conditions, or to make predictions about the ultimate response on a river.

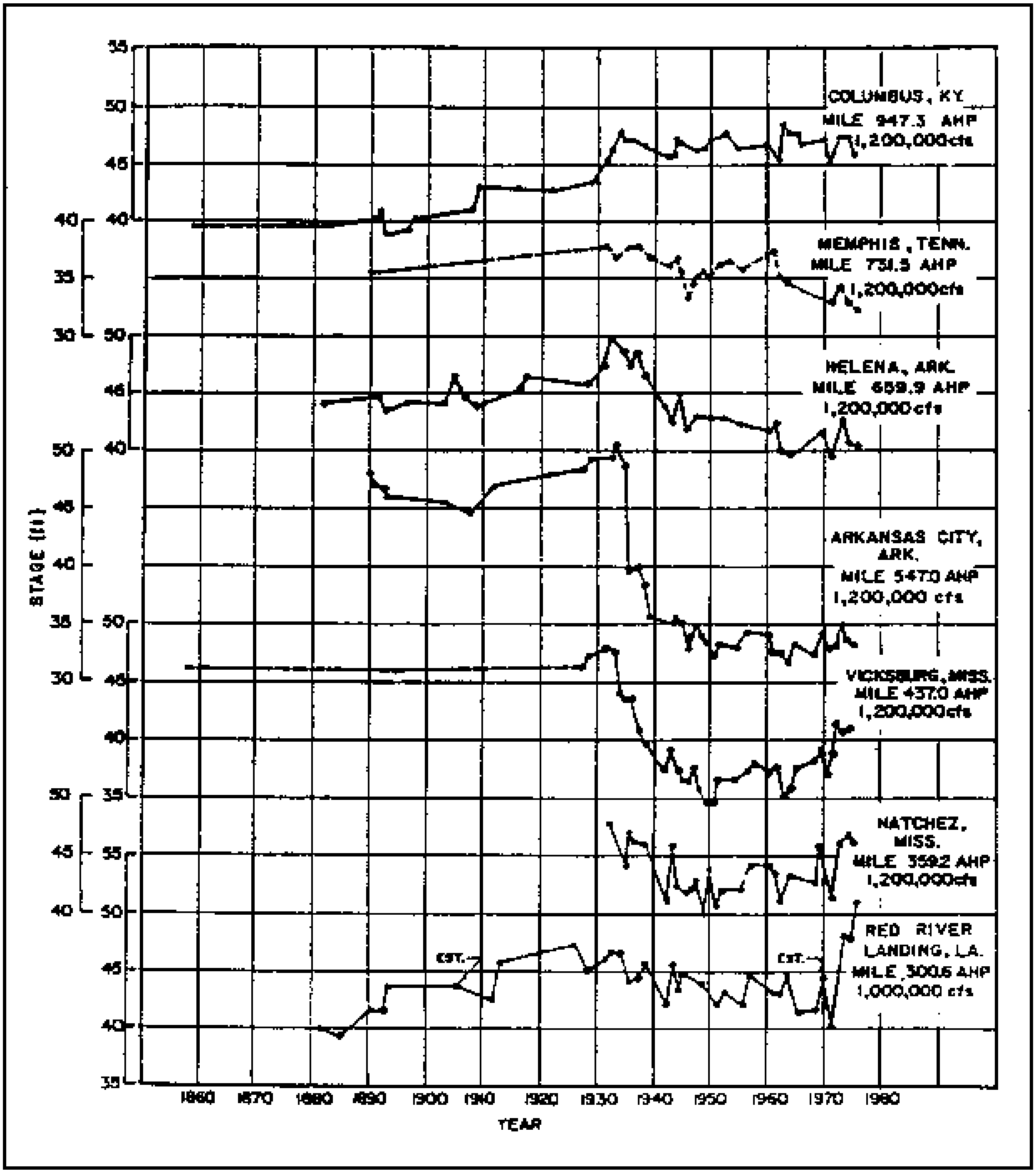


Figure 5.13 Specific Gage Records at Near Bankfull Conditions on the Lower Mississippi River (after Winkley, 1977)

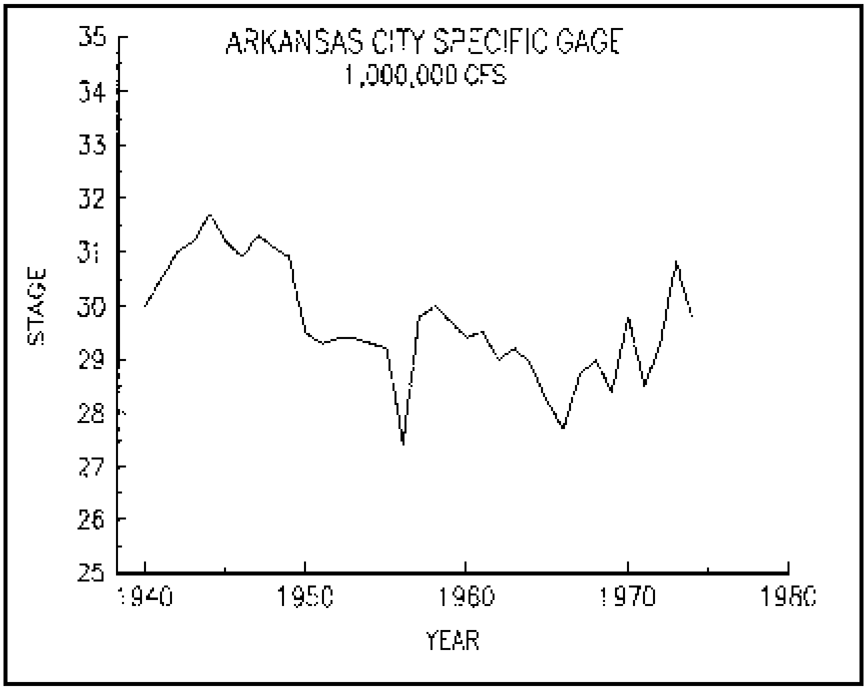


Figure 5.14 Specific Gage Record on Mississippi River at Arkansas City, 1940-1974

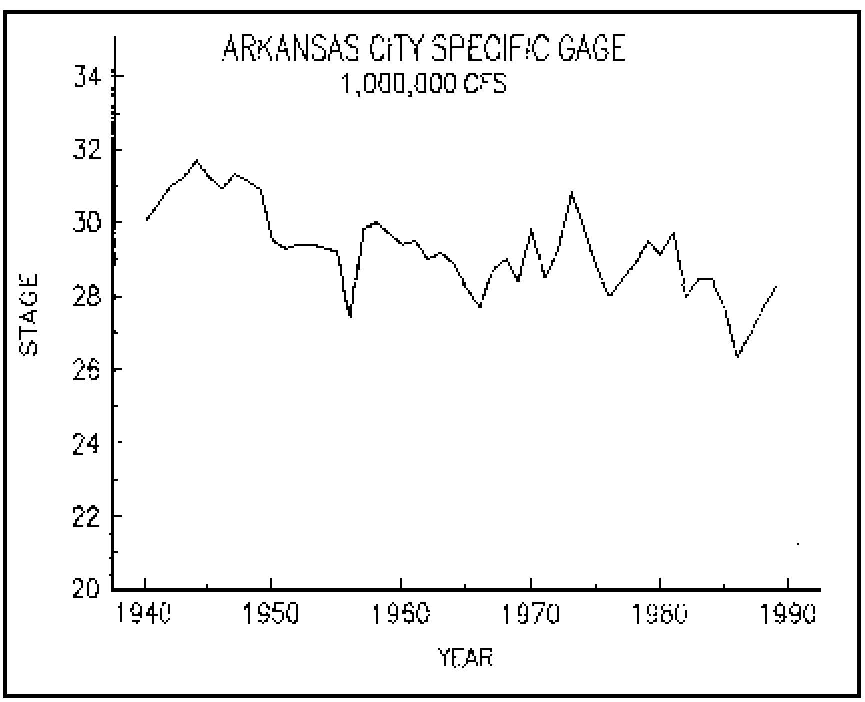


Figure 5.15 Specific Gage Record on Mississippi River at Arkansas City, 1940-1990

5.1.6.2 Comparative Thalweg Analysis

One of the best methods for directly assessing historical channel response is to compare thalweg surveys. This consists of comparing thalweg surveys at different time periods. Comparison of surveys can give a good indication of the historical response of the channel. For instance, a thalweg comparison can show whether the channel bed was degradational or aggradational during the time period between surveys.

Thalweg surveys are taken along the channel at the lowest point in the cross section. Thus, a thalweg survey provides a profile of the bed elevation along the channel at a specific point in time. Comparison of several thalweg surveys taken at different points in time allows the engineer or geomorphologist to chart the change in the bed elevation through time. Whereas, a specific gage record simply reflects changes in the water surface stage, analysis of thalweg surveys can indicate if these changes are due to changes in bed elevation.

The first step in comparing thalweg surveys is to gather all the existing surveys on the channel reach being studied. In most cases, the surveys will be in a cross sectional format. If this is the case then the thalweg elevation must be obtained from the cross section survey. This is not necessary in situations where an actual survey of the thalweg is made by the survey team. The thalweg profiles for each time period are then plotted on the same graph for comparison.

An example of a comparative thalweg survey for Long Creek in north Mississippi between 1977 and 1985 is shown in Figure 5.16. As indicated by this thalweg comparison, the bed of the channel was approximately 10 feet lower in 1985 than in 1977 below about station 320+00. Thus this plot indicates that the channel was degradational at some time during the period 1977 to 1985. However, it provides no information on the stability of the channel bed in 1985. Although the bed was 10 feet lower in 1985 than it was in 1977, this does not necessarily mean that the channel bed was actively degrading in 1985. In fact, it is possible that the channel could have degraded 15 feet between 1977 and 1980, but then began to aggrade after that so that by 1985 the bed was only 10 lower. Therefore, caution must be used when interpreting comparative thalweg profiles. If the surveys are only a few years apart, there may be reasonable confidence that the surveys are depicting what is currently happening in the river system. However, if the time of the surveys are far apart (say 10, 20, or maybe 50 years) then there would be some uncertainty as to whether the surveys reflect the ongoing process.

There are certain limitations that should be considered when comparing surveys on a river system. When looking at thalweg profiles it is often difficult, especially on large river systems, to determine any distinct trends of aggradation or degradation if there are large scour holes, particularly in the bendways. These local scour holes may completely overwhelm the variations in the thalweg. For instance, on a large river system such as the Mississippi River, scour depths may be in excess of 60 feet, but variations or changes in the overall bed elevation may be an order of magnitude less. In this situation, it would be very difficult to note any aggradational or degradational trends because of the scale effects. This problem can sometimes be overcome by eliminating the pool sections, and focusing only on the crossing locations.

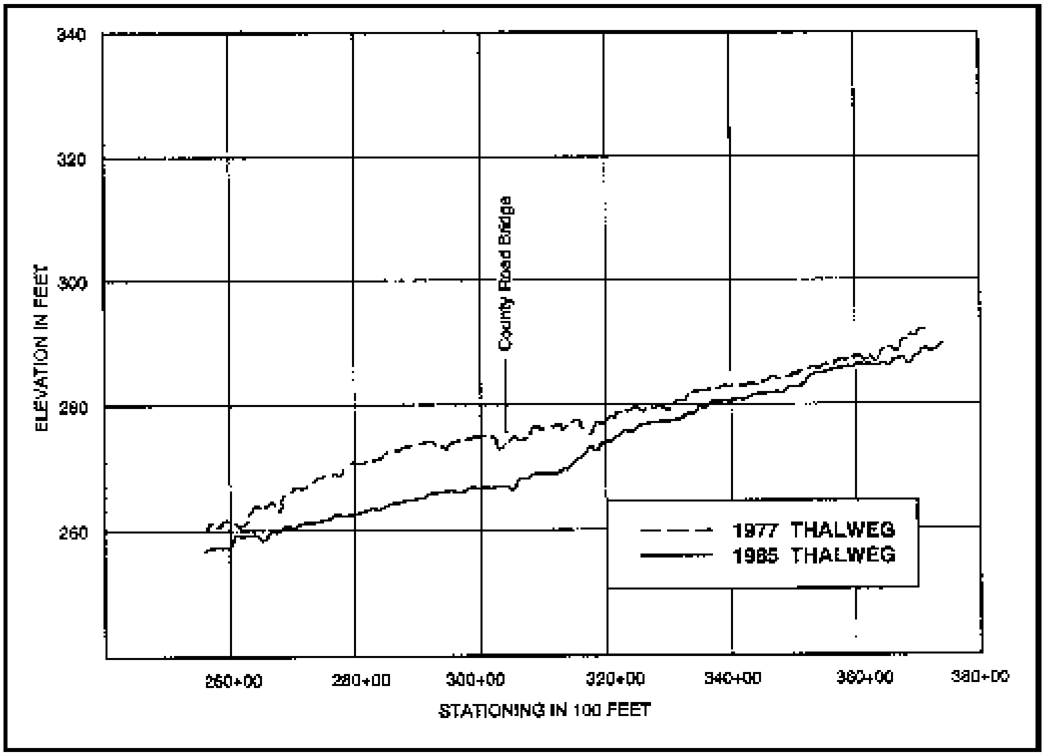


Figure 5.16 Comparative Thalweg Profiles for Long Creek, Mississippi

Focusing on the crossing elevations may eliminate much of the local variation due to bed scour in bendways, thereby, allowing aggradational or degradational trends to be more easily observable.

5.1.6.3 Analysis of Cross Section Geometry

Section 5.1.6.2 addressed the use of comparative thalweg profiles to assess the stability of a channel system. While thalweg profiles are a useful tool it must be recognized that they only reflect how the behavior of the channel bed and do not provide information about the channel as a whole. For this reason it is often advisable to study the changes in the overall cross sectional geometry. Cross sectional geometry refers to width, depth, area, wetted perimeter, hydraulic radius, and channel conveyance ($AR^{2/3}$) at a specific cross section.

If channel cross sections are surveyed at permanent range locations then channel geometry, as reflected by depth, width, area, width-depth ratio, and conveyance (usually calculated as $AR^{2/3}$) can be compared directly at different time periods. At each range, the cross section plots for the various time periods can be overlaid and compared.

One problem with the above method is that there may be so many cross sections that it becomes impossible to interpret the results. Another problem is that it is seldom the case that the cross sections are located in the exact same place year after year. Because of these problems it is often advisable to compare

reach average values of the cross sectional geometry parameters. This requires the study area to be divided into distinct reaches based on geomorphic characteristics. Next, the cross sectional parameters are calculated at each cross section, and then averaged for the entire reach. Then the reach average values can be compared for each survey period.

A simple example of the use of comparative surveys to document historical channel changes can be seen by observing the behavior of the Simmesport discharge range on the Atchafalaya River in Louisiana at six points in time between 1931 and 1977 (Figure 5.17). As shown in Figure 5.17, the channel at this location has been progressively deepening and widening since the early 1930s.

The above example is a very simple case where the channel changes at a specific location were analyzed. However, the response at this particular location may not be representative of the morphology of the rest of the channel system. For this reason, it is usually best to compare cross sections along the entire length of the study reach. Unfortunately, it is very rare for cross sections to be located in the same place from survey to survey. This presents a problem in making direct comparisons of cross section dimensions. One solution to this problem is to develop and compare reach average values of cross sectional parameters. Biedenharn (1983) used reach average values to document the channel response on the Little Tallahatchie River below Sardis Dam in north Mississippi. The study area below Sardis Dam was divided into two geomorphic reaches. Reach 1 extended from the dam to Floyd's Island, a distance of 2.7 miles. Reach 2 extended about 7 miles below Floyd's Island to the Railroad Bridge. Channel surveys were available for the study area from 1940 to 1980. Unfortunately, the locations of the cross section ranges varied somewhat on each survey. Therefore, the average depth, width, and area from all the cross sections in each study reach were calculated. Figures 5.18 and 5.19 show the average cross section geometry changes from 1940 to 1980 for reach 1 and reach 2, respectively. These reach averaged values were used in conjunction with specific gage analysis to document the morphologic response of the channel to the construction of Sardis Dam.

5.1.6.4 Aerial Photography

A comparison of historical and present aerial photography can identify areas that have been channelized or realigned. Aerial photography also provides some information on land use changes. With good quality photography, it is possible to locate knickpoints in the channels as well as areas of instability (bank caving and channel widening) and their progressive growth.

Vertical changes in the river system are also determined using stereoscopic means. This is especially useful in a large river system where vertical changes in the point bars or middle bars can be identified. The average height of the point bars or middle bars can be measured for different time periods using stereoscopic pairs. A decrease in the overall height of the bars indicates a degrading river system. The level of confidence of these analyses depends on the time between surveys.

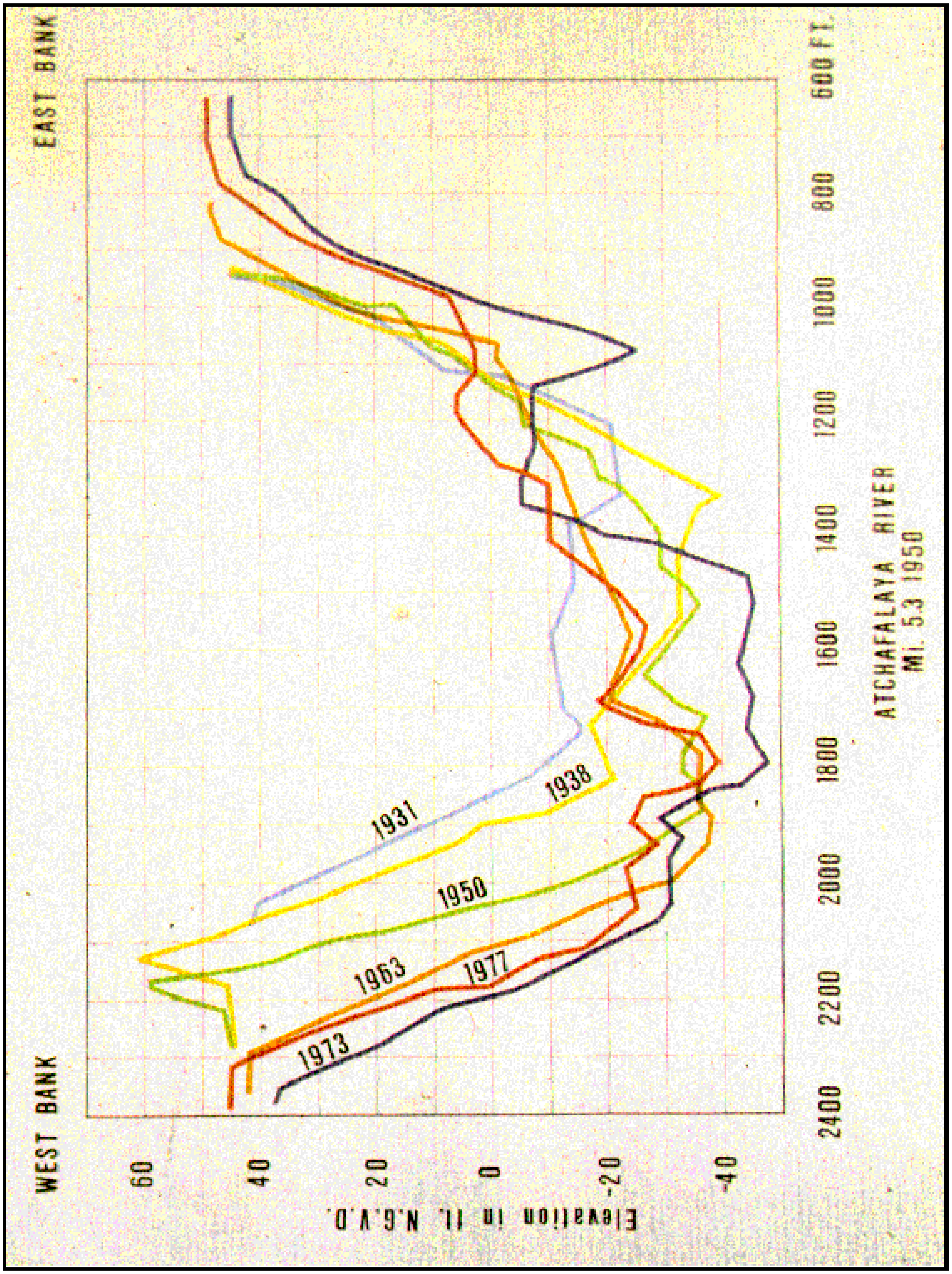


Figure 5.17 Cross Sectional Changes on the Atchafalaya River at Simmesport, Louisiana

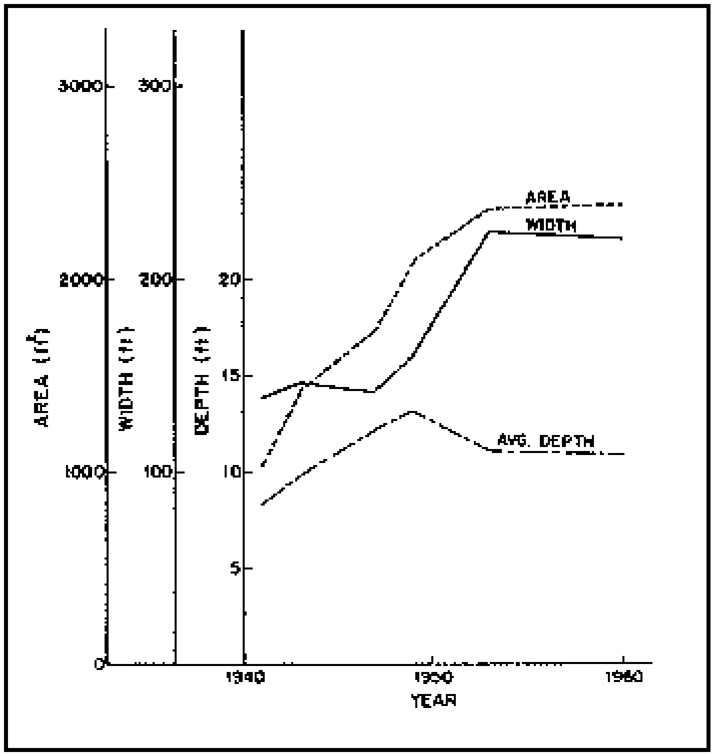


Figure 5.18 Average Cross Sectional Values for Little Tallahatchie River Below Sardis Dam

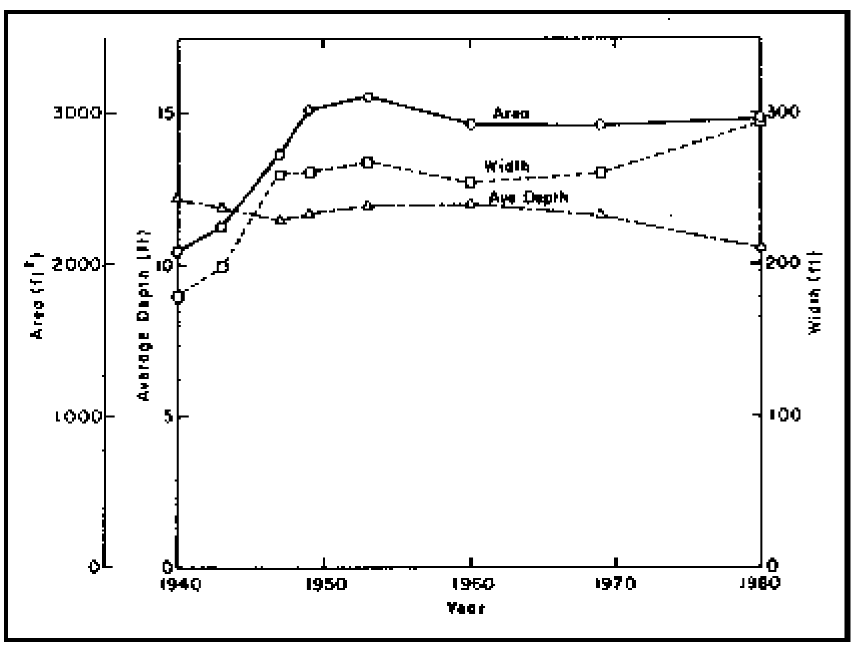


Figure 5.19 Average Cross Sectional Values for Little Tallahatchie River Below Sardis Dam, Reach 2 (after Biedenharn, 1983)

5.2 FIELD INVESTIGATION

One of the most important aspects of any study of watershed geomorphology or channel stability are field investigations. Data critical to understanding the physical state of the project area can be obtained from thorough field studies. Field work can be divided into quantitative and qualitative studies. The quantitative aspects of field work involves site surveys, hydraulic measurements, estimate of channel roughness, soil sampling, and sediment sampling. Channel surveys are designed to provide information for determining average channel geometry, channel slope, and bed and bank profiles. Cross-section surveys are often taken at intervals based on channel width. The interval length depends upon economics, position of controls, dimension and character of channel related failures, sediment characteristics, and channel configuration. Within the economic constraints of the project, surveys must be extensive enough to accurately represent the channel and the pertinent features. The survey sections should also sufficiently extend beyond the top of the banks to record the general level of the immediate floodplain. In addition to the cross-sectional surveys, the water surface and bank profiles are surveyed to determine the slope of the study area.

Hydraulic measurements such as average water surface width, stage, and water temperature supplement the channel survey data. In addition, long-term measurement of flow velocity and subsequent discharge calculations at selected channel cross sections provide valuable hydraulic historical data.

An estimate of the roughness coefficient should be determined for the left overbank, right overbank, and channel. Roughness coefficients are a function of surface roughness, amount of vegetation, channel irregularities, and to a lesser degree, stage, scour, deposition, and channel alignment. These coefficients will be used in numerical models for computing water surface profiles, sedimentation, and channel stability. It is important to recognize that roughness coefficients may vary dramatically from left overbank to right overbank and even within the channel. These variations can significantly model calculations and care should be taken to ensure that accurate estimations are made. If the stage, discharge, and slope are known at a given cross section, the roughness coefficient can be calculated. Chow (1959) outlines Cowan's approach to estimating Manning's n. Two excellent pictorial references are estimating the roughness coefficients using a visual comparison method are: *Roughness Characteristics of New Zealand Rivers* (Hicks and Mason, 1991) or *USGS Roughness Characteristics of Natural Channels* (Barnes, 1989).

Soil samples should be collected from the bed and banks and analyzed to determine geotechnical characteristics such as unit weight, angle of repose, angle of internal friction, cohesion, and soil particle size. If the bank is stratified or layered, a sample should be taken from each layer. Edwards and Glysson (1988) define bed material as the sediment mixture of which the bed is composed. Descriptions of bed material sampling can be found in Julien (1995), Edwards and Glysson (1988), or Petersen (1986). For coarse bed material, Hogan (1993) describes several procedures that can be performed to determine the bed material composition. This information is used for the stable channel design computations and slope stability analysis.

The dominant sediment transport mechanism is primarily based on the soil and sediment characteristics. Measurements of suspended load or bed load sample should be taken upstream of channel and bank failure sites to estimate the sediment moving through a stable system. The accuracy of sediment sampling techniques, however, are often limited. Julien (1995), Edwards and Glysson (1988) or Petersen (1986) provide guides to field methods for measuring fluvial sediment. Alternatively, sediment transport can be estimated using empirically based sediment transport capacity equations. Sediment transport can be divided into three zones that describe the dominant mode of transport: bedload, mixed load, and suspended load (Julien, 1995, p. 186).

5.2.1 QUALITATIVE OBSERVATIONS

The qualitative portion of a field investigation is an integral part of the overall assessment process. Field observations should be recorded in an organized fashion on site assessment sheets which detail all pertinent site characteristics. The sheets divide the description into channel, bed and bank investigations. Thorne (1992) describes the use of evaluation sheets as an aid to field identification of the following channel characteristics:

1. the state of vertical and lateral channel stability;
2. the relation of local bank retreat to channel instability;
3. the engineering and morphologic characteristics of the banks;
4. the dominant erosive forces and processes;
5. the state of bank stability and the major failure mechanisms; and
6. the input parameters necessary for modeling bank retreat.

The sheets are designed to provide a systematic and disciplined approach to the collection, recording, and interpretation of both archive and field data.

5.2.2 SKETCHES

Detailed cross sectional and planform sketches of the study reach should be made to supplement the observations. The sketches should identify and locate the relative positions of:

1. the type of flow conditions;
2. bed and bank controls;
3. dominant bed materials and bed forms;

4. significant morphological features;

5. nature of bank materials and evidence of instabilities;

6. vegetation; and

7. structures.

5.2.2.1 Field Identified Features

The typical types of field-identified features include:

a. Knickpoints/Knickzones. As discussed in Chapter 3, channel degradation is the result of an imbalance in the sediment transport capacity and supply. A field indication of degradation occurs in the form of knickpoints or knickzones. These are often referred to as headcuts. However, there is considerable confusion in the terminology. According to Schumm *et al.* (1984) a headcut is defined as a headward migrating zone on incision. A knickpoint is a location on a thalweg profile of an abrupt change of elevation and slope. A steeper reach of channel representing the headward migrating zone is referred to as a knickzone.

b. Berms. The formation of berms can indicate an attempt by a channel to establish stability. Berms from after a channel has degraded and channel widening and slope flattening have progressed to the point where the sediment transport capacity is reduced. This in turn reduces the hydraulic removal of failed bank material at the toe of the bank and also allows sediment deposition to occur at the toe of the bank. The stability of berms is improved after vegetation (particularly woody species such as willow, river birch, and sycamore) is established. Berms may be associated with the incision channel's development of a new floodplain.

c. Terraces. A terrace is another feature that provides information on channel behavior. Terraces are erosional features resulting from bed lowering, while berms are depositional features which form as the channel regains stability following bed lowering. When a channel degrades, it leaves an erosional escarpment which was previously the top bank. This is called a terrace or inactive floodplain. Therefore, terraces are indicators of past degradation in a channel. The tops of terraces are usually much higher than the active floodplain and may only be overtopped by extreme flood events.

d. Sediment Sources/Samples. Major sediment sources to the channel are recorded during the field investigation. These sources include the bed and bank of the channel, tributaries, gullies, drainage ditches from roads and highways, and watershed (upland) erosion. In the degrading channels, the major sources of sediment are the deteriorating channel banks and beds. In this case, the sediment is introduced into the system over a sometimes lengthy reach of channel. In contrast, tributaries that are undergoing similar instabilities may be points of concentrated sediment input. During the field investigation, note the amount of sediment deposited at the downstream of the confluence of tributaries.

Sediment sampling provides information on the composition of the sediments derived from each source. In general, the channel bed material samples are taken at the thalweg in order to obtain a representative sample. Analysis of these samples provides information on the spatial variations of grain size within the channel system. Samples of channel bank material and, if applicable, each stratigraphic layer, are collected. Sediments in tributary mouth bars are used to determine if tributary sediments are radically different from the channel sediments. Samples taken at surveyed cross sections can be correlated to the channel hydraulics, geometry, and geomorphology.

e. Sediment Depth. The depth of sediment in the channel bed can be useful in determining the stability status of the channel bed. For most streams, an average sediment depth of 3 to 4 feet or greater is an indicator of an excess sediment supply and, hence, aggradational tendencies for the reach. Likewise, an average sediment depth of 1 foot or less indicates an excess of transport capacity and possible degradational tendencies for the reach.

Depth of sediment is easily determined in the field by probing the channel bed with a metal rod. Probing indicates the presence of clay outcrops or coarse material below the surface, and is done frequently along a reach to find the average sediment depth. Although probing of point bars or even berms can be beneficial, probing should be concentrated at the channel thalweg. Correlating sediment probing with survey cross sections is recommended.

f. Bank Heights and Angles. Heights and angles of the channel banks are field-determined to assist in the geotechnical stability assessment. These data can be determined from survey cross sections, but field verification is recommended since survey cross sections may not be representative of the entire reach. Bank heights and angles are used to establish geotechnical stability criteria for the channel reach. Field measurements include measurement of bank height with a survey rod or cloth tape, and of bank angle with an inclinometer. Measurement at locations where bank failure is impending or has recently occurred is a bonus. These measurements are used to empirically define stability criteria for the channel reach.

Also, observe tension cracks in the upper bank and mode of bank failure. Tension cracks can indicate a stressed condition in the upper bank which can lead to slab type failure. Slab failure is the failure due to gravity of large mass blocks of the upper bank along a near vertical plane. The classic rotational failure is rotation of the bank mass along a circular arc.

g. Bank Stratigraphy. Proper identification of bank stratigraphy and its role in channel bank stability is probably best determined by an investigator with a background in geology. A classification of the general composition of the observed layers and the percent of the total bank composed by each layer are made in the field investigation. If the strata indicated bank instability, then the field data can be analyzed by a geologist at a later date.

h. Vegetation. The spacial distribution, size, and approximate age of the vegetation existing within and along a channel are recorded in the field investigation. Vegetation colonizing with the channel and along berms are evaluated with respect to growth and whether or not it may be removed by the next flood flow.

Substantial in-channel vegetation along the berm indicates lateral stability in the channel. In-channel vegetation may impact the hydraulic efficiency of the channel.

Depending on the degree of channel incision, top bank vegetation may or may not contribute to bank stability. Highly incised channel banks may not benefit from the erosion resistance offered by root systems, and may even be overburdened by the weight of the trees. Streams which have numerous toppled trees and other woody vegetation in the channel may have recently had an episode of degradational instability.

i. Hydrologic Features. During the field investigation, estimates of Manning's n values are made for the various reaches of the channel. These data are important for computing water surface profiles in subsequent phases of the investigation. Roughness (n values) is determined for the active channel, the berms, and the floodplain.

Vegetation frequently preserves evidence of water surface elevations during floods. Debris transported during floods is often trapped in the vegetation. These high water marks are recorded at the surveyed cross sections, even if the method of measurement is crude. High water marks are also used to calibrate n values.

Any evidence of frequent overbank flows such as sand splays, overbank erosion, and crop damage, etc., are noted during the field investigation. These areas may need consideration for flood control measures during formulation of the watershed plan.

j. Existing Structures. The presence of man-made features, the extent of the features, and their location along the channel is recorded on the aerial photos. Man-made features include bridges, bank protection sites, drop inlet structures, culverts, and grade control structures. An assessment of the effectiveness of the various features is made during the field investigation. Evidence of scour on bridge pilings and culverts provides information on the amount of degradation that has occurred since the construction of the structure. The overall effect of channel stability on the basin infrastructure is assessed.

5.2.3 CHANNEL, STREAMBED, AND STREAMBANK DESCRIPTIONS

The channel description characterizes the stream channel and the adjoining area. The study area as well as the reaches just upstream and downstream of the site should be the main focus of the field investigations. As much information as possible should be collected (within the financial constraints of the project) to accurately analyze the fluvial processes occurring.

The following terminology is used to describe channel, streambed, and streambank characteristics:

Terraces are fluvial landforms produced by past vertical instabilities in the fluvial system. Leopold *et al.* (1964) define a terrace as an abandoned floodplain not related to the present stream. The sequence of events leading to the observed features in the field may include several periods of alluvial deposition. If incision and aggradation occur repeatedly it is possible to develop several

terraces. Figures 5.20 and 5.21 illustrate stages of terrace development. The presence of terraces as well as the number should be noted.

Overbank deposits describe the presence and amount of material deposited directly onto the valley floor by out-of-bank flows. The magnitude of overbank deposits belies the sediment transport capabilities of out-of-bank flows. Deep, fast overbank flows are usually indicative of active floodplain processes which are usually associated with aggrading streams.

Trash lines are remains of floating trash and vegetation left after a flood flow recedes and often provides a good indication of the high water mark. Most often the debris is found in the floodplain and can usually be found attached to trees and bushes. Trash lines degrade quickly once the flood flows recede. The frequent appearance of trash lines resulting from flow rates with short recurrence intervals suggests that the stream may be aggrading.

Adjacent land use describes the type of activity or land modifications taking place in the areas adjacent to the site. Generally, cultivated areas have higher runoff potential and sediment yield than natural settings. Urban and suburban catchments produce flashy runoff hydrographs and extremely varied sediment yield.

Riparian buffer zone and *strip width* describe the presence of natural vegetation buffer zones along a stream. The riparian zone provides several important ecological functions such as providing wildlife habitat, intercepting surface runoff, reducing sediment yield, providing a sink for pollutants in surface and subsurface flows, reducing near bank velocities, reinforcing bank materials, and limiting access to the bank by grazing animals.

Flow type defines the regime of flow in the stream at the time of observation. Flow type is a function of bed forms and bank material and is highly dependent on the stream gradient. Grant *et al.* (1990) developed a relationship between bed forms and gradient (Figure 5.22). Uniform/tranquil flow is characterized by uniform flow velocities and channel characteristics. Uniform/rapid flow involves significant changes in velocity along the channel. Pools and riffles generally are seen at low flows and represent a flow regime that alternates between shallow and deepened features which produce non-uniform flows. Pools are areas of deep, slower moving flow with a gentle water surface slope that generally result from localized scour. Keller and Melhorn (1973) distinguish two types of pools in meandering channels: primary and secondary. Primary pools, which exhibit deep scour, are usually found at bends and are typically associated with point bars. Secondary pools, which are scoured to a lesser depth than primary pools, are not necessarily associated with point bars. Riffles are shallow areas characterized by fast moving flow which results from bed material deposition. Steep and tumbling flows occur in high gradient streams with coarse bed materials. These flows produce localized supercritical flow between and over boulders. Steep and step/pool flow is found in very steep channels with boulders or woody debris arranged in periodic steps across the channel and plunge pools in between.

Bed controls describe the presence of local geology, materials, or human structures that resists being eroded by river processes and thereby controls vertical instability.

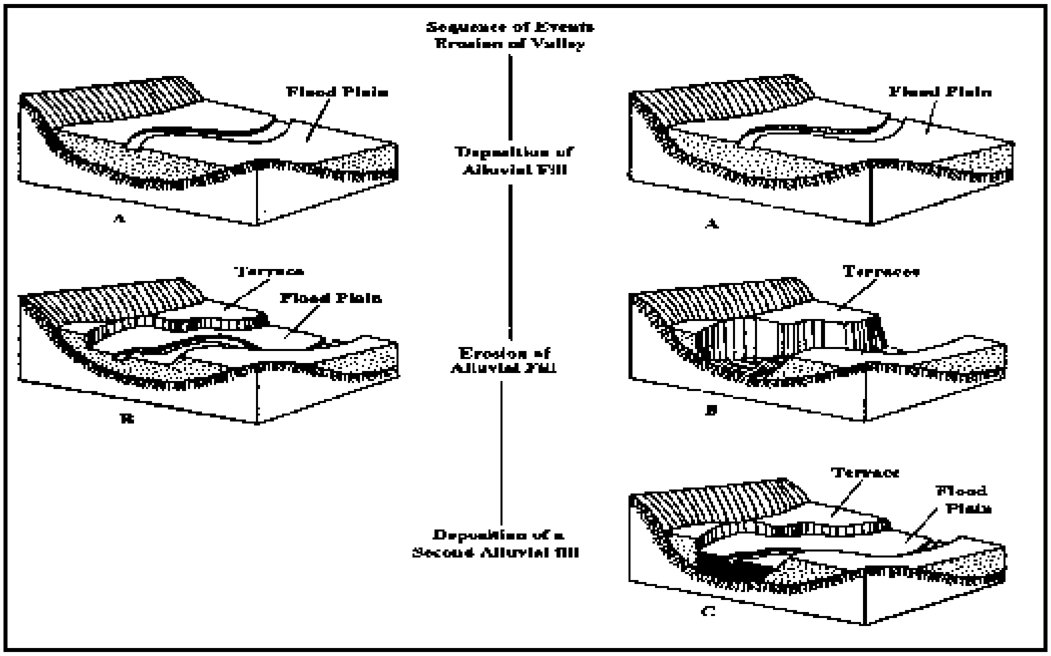


Figure 5.20 The Stages of Terrace Development Following Two Sequences of Events Leading to the Same Surface Geometry (after Leopold *et al.*, 1964)

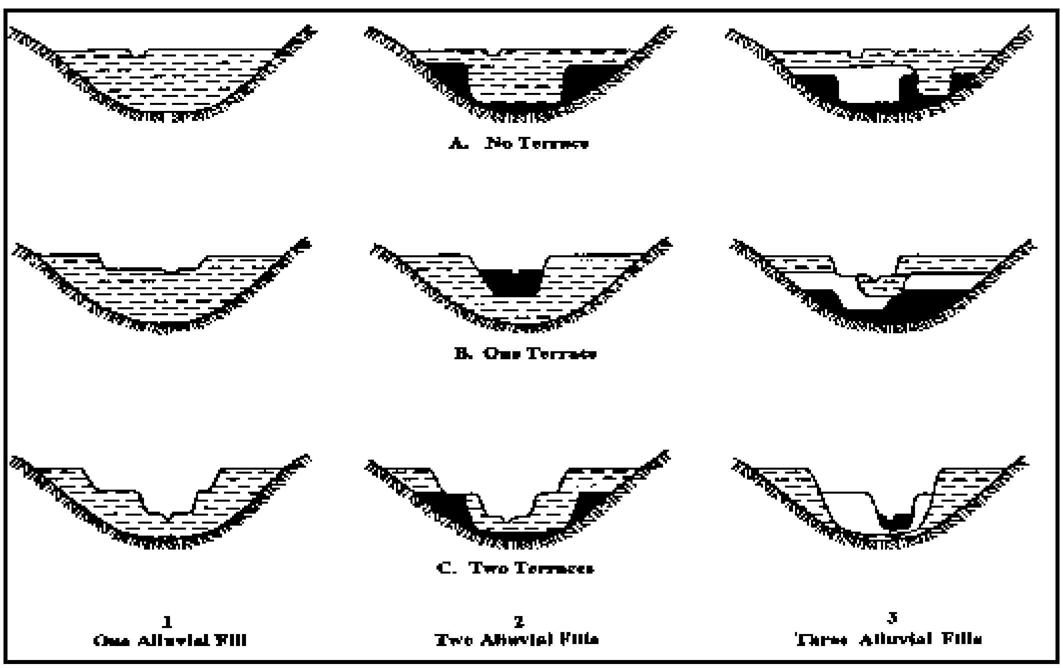


Figure 5.21 Examples of Valley Cross Sections Showing Some Possible Stratigraphic Relations in Valley Alluvium (after Leopold *et al.*, 1964)

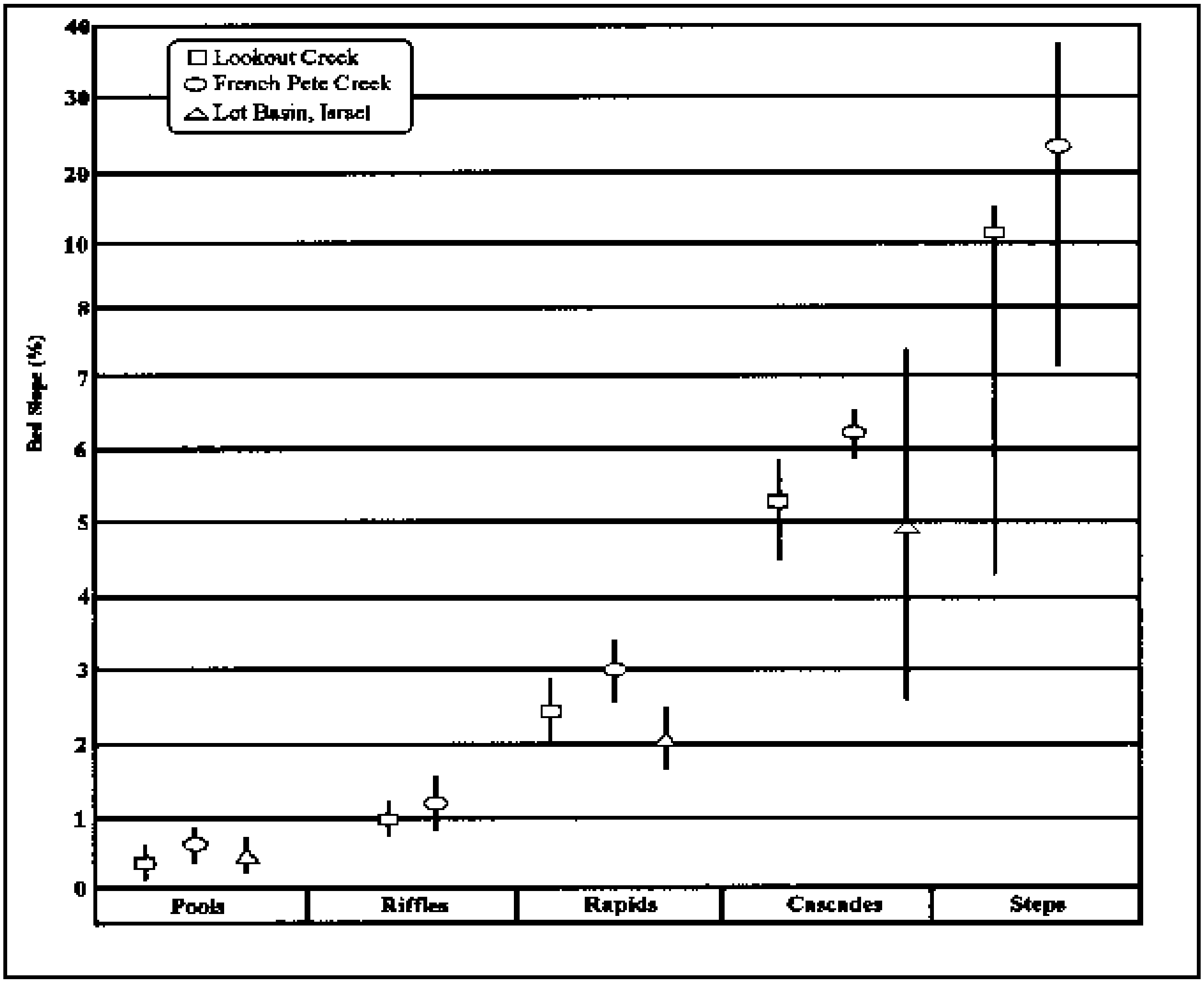


Figure 5.22 Relationship Between Gradient and Bed Forms (after Grant *et al.*, 1990)

Control type describes the nature of the bed controls. Natural examples include bedrock outcroppings, coarse sediments that form a layer of immobile armoring, or fine sediments that are strongly cohesive. Weirs or other grade control structures are examples of man-made structures which function as bed controls.

Bed material is the bed sediment of the river. The bed material description is very important when determining the flow resistance.

Islands or *bars* are bed features that have significant effects on flow resistance, channel capacity, and in-channel sediment storage.

Bar type describes the shape or type of bars present in the reach. Pool/riffle bars are alternating features that run across the width of the channel and are visible at lower flows. Riffles

are followed by a pool of deep, slower moving water. Alternating bars, which are accumulation of sediments positioned successively downstream on opposite sides of the stream, are generally found in straight sections of the river. Sediment accumulations on the inside of the meander bends in meandering streams are described as point bars. Given enough time, alternating bars will become point bars if the stream is allowed to meander. Mid-channel bars and islands are generally associated with braided streams. Braided channels develop in streams that have an abundant bed load, erodible banks, a highly variable discharge, and high stream power (Knighton, 1984, pp. 144-146). The different bar types are presented in Figure 5.23.

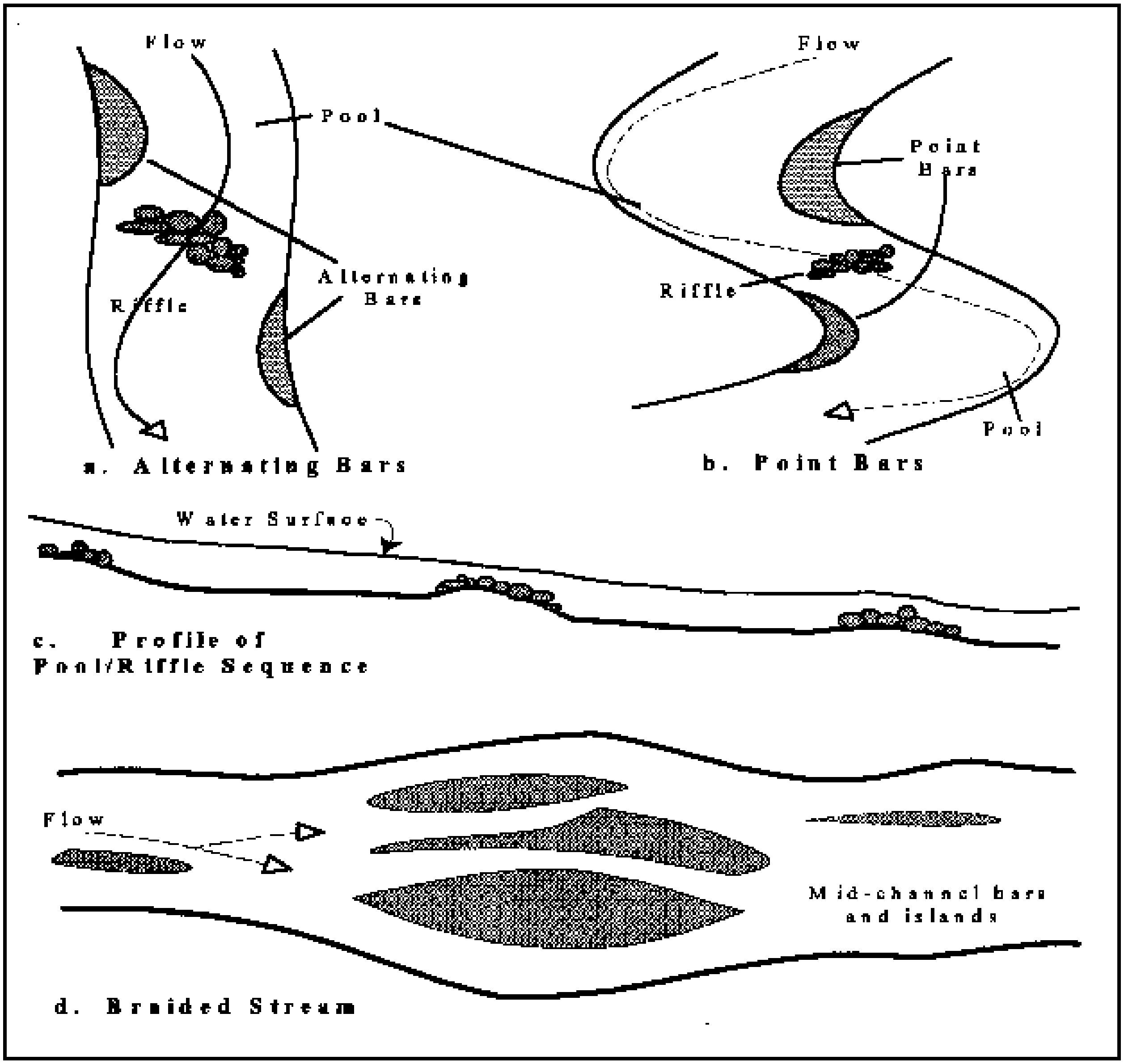


Figure 5.23 Bar Types

Bed forms note the presence and type of bed forms that may develop in sand bed channels. The primary variables that affect bed forms are the slope of the energy grade line, flow depth, bed particle size, and particle fall velocity (Julien, 1995, p. 138). Julien (1995) or Simons and Sentürk (1992) provide more extensive information on bedform classifications and prediction. Figure 5.24 presents the basic bed forms.

Bed armoring refers to the presence of a coarse surface layer on the streambed. In noncohesive

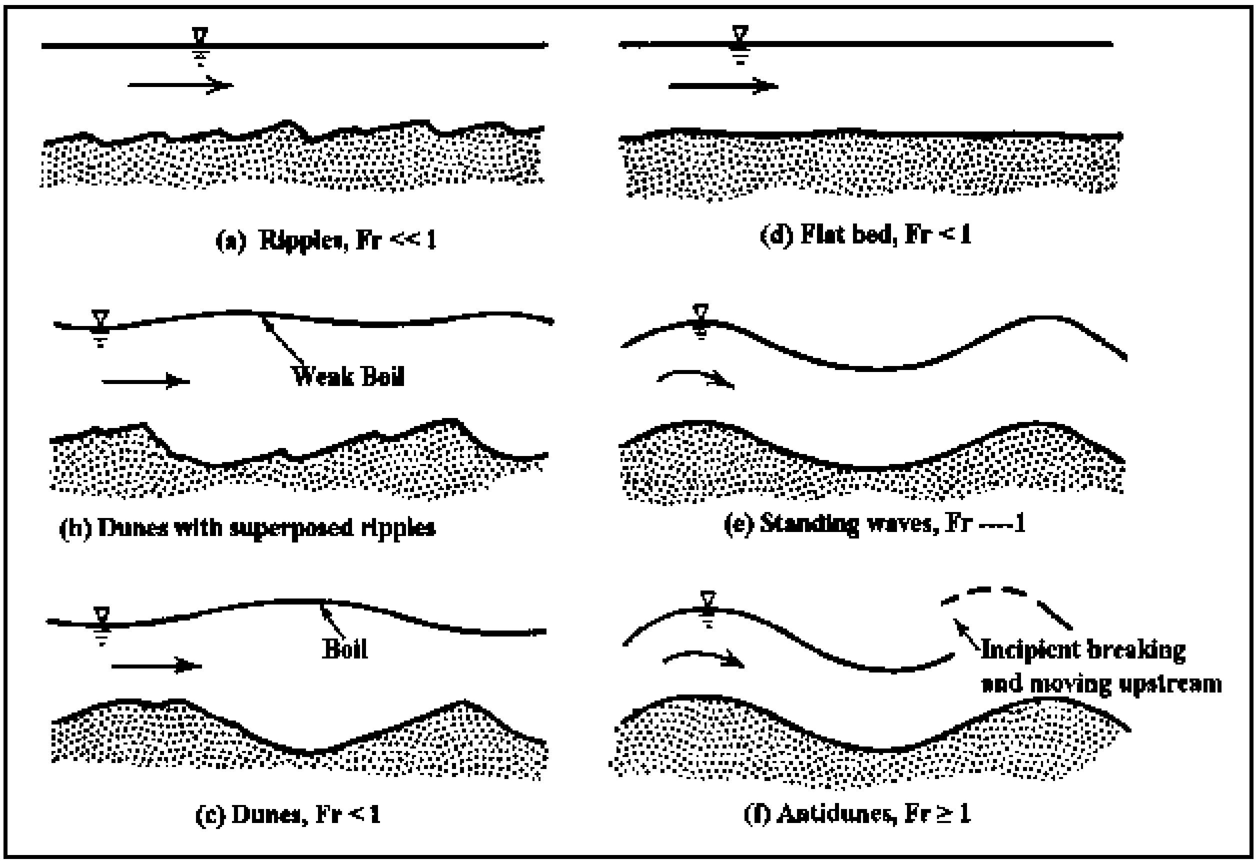


Figure 5.24 Bed Forms (after Simons and Richardson, 1966)

sediments, the materials available for transport are essentially those exposed at the bed surface. The active layer refers to the surface layer from which materials can be entrained by the flow. Below that may be one or more layers of immobile coarse sediment in which the majority of the finer sediments have been scoured away. This layer is known as armoring and protects the underlying material from further scour (Chang, 1988, p. 177).

Signs of aggradation or degradation note the presence of features that usually are indicative of vertical instability. Headcuts or knickpoints (Figure 5.25) are defined as locations on the streambed profile where there is an abrupt change of elevation and bedslope (Schumm *et al.*, 1984, p. 9). The headcut is an adjustment by the river to restore equilibrium in the system. Headcuts often migrate upstream along the channel incision and increased sediment transport.

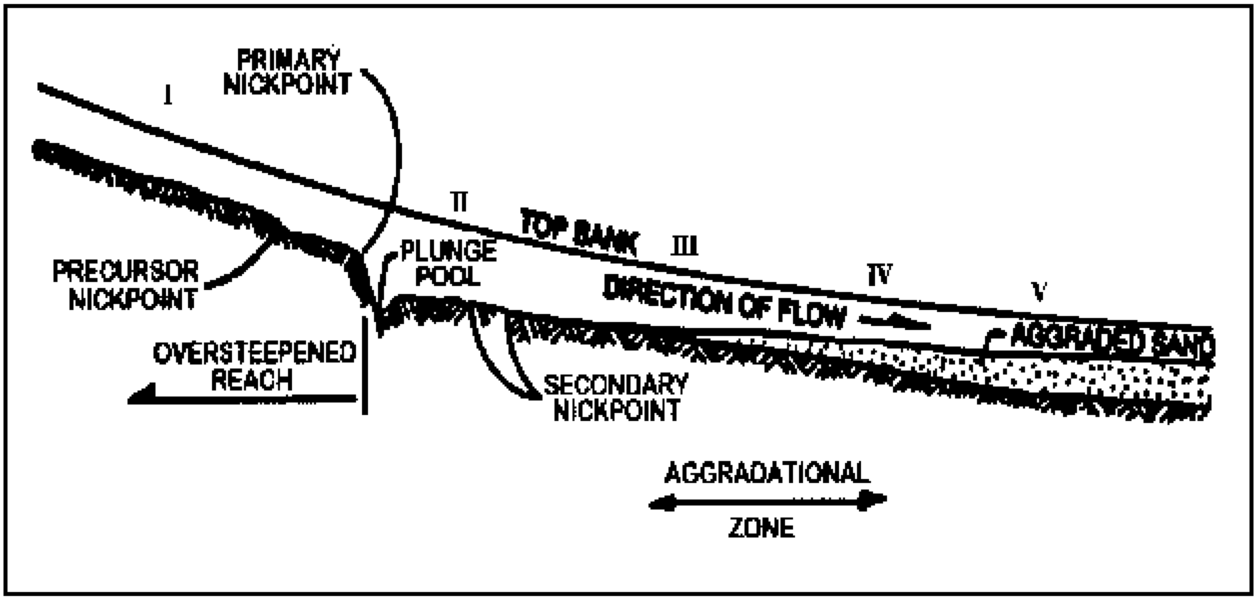


Figure 5.25 Headcuts (after Schumm *et al.*, 1984)

Perched tributaries, which are sharp changes in bed elevation at stream junctions, are an indication of a degrading streambed. If the main channel is incising while the tributary is not, there will be a sharp difference in bed elevations where the two channels meet.

The depth of loose sediment deposits on the bed, alternating bars in the channel, and frequent overbank flows generally indicate an aggrading stream. Degradation of structures and widespread bank failures are also good indicators of degradation.

5.2.4 BANK CHARACTERISTICS

The bank characteristics analysis examines soil types, vegetation, the presence of bank structures, erosion processes, and geotechnical failure mechanics. The bank angle and bank height can best be determined by field survey; however, during field reconnaissance these values can be estimated. The following terminology is used to describe bank characteristics:

Soil types describe the classification of the bank materials as cohesive, noncohesive, composite or layered. Sower (1979) describes cohesive soils as soils for which the absorbed water and particle attraction work together to produce a body which holds together and deforms plastically at varying water content. Cohesive banks are typically formed in silts and clays while noncohesive banks are formed from sands, gravels, cobbles, and boulders. Composite banks consist of layers of cohesive soils intermixed by layers of non-cohesive soils. Streams flowing through alluvial deposits often have composite banks. Non-cohesive deposits are relics of former channel bars and deposition zones that become covered by silt and clay deposits. The interface between the cohesive and non-

cohesive layers is usually distinct and well defined (Thorne, 1981, p. 460). The number of layers present should be noted as well as the thickness of the layers.

Tension cracks and *crack depth* note the presence of tension cracks. Generally, tension cracks develop vertically down the bank face on steep banks and significantly reduce the stability of a bank with respect to mass failure. The width and depth of the crack should be recorded.

Structure type describes the presence and type of any structure in the study reach.

Structure condition describes the current condition of the structure. A stable designation indicates a structure that is functioning as designed and is not being undermined or destroyed by unstable fluvial processes. Marginal describes a structure that is not completely functioning as designed or is degrading or near failure. Failed characterizes a structure that is not performing as designed or has failed due to some adjustment in the fluvial system.

Observed problems identify the type of problems related to the structure and the failure to perform acceptably.

Bank vegetation describes the type, condition, and location of vegetation. The general types of vegetation prevalent in and along the streams are important for determining the overall bank stability, the rate of bank shifting, the erodibility of the banks, and the resistance to flows. The existence of vegetation on the bank can serve as an indicator of bank stability.

Vegetation broadly classifies the types of vegetation along the bank.

Tree types describe the different types of trees along the bank. Different tree types affect the bank stability in different ways. Conifers are shallow rooted and lack a thick vegetative cover compared to deciduous trees. Leaning trees are an excellent indication of an upcoming mass bank failure and the angle should be noted. High water level can be estimated as the level at which tree growth begins.

Health describes the condition of the vegetation. Dead or dying vegetation can be a serious liability to bank stability.

Roots describe the relationship between the vegetation roots and the bank surface. If the bank surface is relatively stable, the roots are normally found just below the surface. If sediment is accumulating on the bank, vegetation produces adventitious roots into the new sediment. If the bank is eroding, roots are exposed. If the erosion is rapid, the roots are standing straight out of the bank face, while if the erosion is gradual, the roots often turn and grow back into the soil.

Height is an important factor in determining the effects of vegetation on impeding near bank flows. Tall vegetation encourages sedimentation while it reduces conveyance. Note the height of the average vegetation.

Diversity describes the mixture of vegetation types present. Diversity is directly related to age. Generally, a mature system has a number of different species.

Density and *spacing* describe the degree of vegetative cover of the bank face from a visual inspection. Density refers to the vertical thickness of vegetation. The denser the vegetation the better the erosion protection and the greater the resistance to flow. Spacing describes the location of vegetation across the bank. Clusters refer to vegetation with gaps in coverage that flow can attack while continuous describes complete coverage along the bank.

Age estimates the age of the vegetation. Age estimations are used as a guide to the history of the bank. Mature vegetation can only develop on a stable bank, while a predominance of young, immature vegetation gives some insight into the recent history of the area. Estimating the age of vegetation requires significant experience, but an approximation can often be made based on the size and height of the vegetation.

Bank toe accumulation characterizes the balance between the sediment supply and sediment removal at the toe of the bank. Banks that have net toe erosion become less stable with time. Banks that have neither net toe erosion or deposition continue to erode at about a constant rate as eroded materials are transported at the same rate as generated. Banks with net deposition generally demonstrate greater stability. With time, vegetation will colonize sediment deposits at the toe.

Stored bank debris notes the presence and type of material found in storage at the bank toe. The material should primarily be derived from the bank and not an accumulation of bed sediments, which would indicate a bar.

Vegetation and the *vegetative characteristics* at the toe can give a good idea of the toe sediment balance. A stable sediment shelf will have mature vegetation while a newer deposit may only have young, immature vegetation. Roots will be exposed on a toe deposit that is eroding and adventitious roots will be present for depositional zones.

Bank erosion describes the processes that lead to hydraulic failure or the detachment and transportation of individual grains. The purpose of this section of the form is to identify the processes responsible for the erosion and the distribution of these mechanisms along the banks of the study reach. Gray and Sotir (1996), Goldman *et al*. (1986), Petersen (1986), and Gray and Leiser (1982) give more complete explanations of surface erosion mechanisms.

Erosion location establishes the position of the eroding section in relation to major channel features. It is important to note the location of erosion in relation to channel planform, bed features and engineering structures. The field sketches and surveys are very important in providing this information.

Processes attempt to identify the mechanisms responsible for the hydraulic failure. As mentioned earlier, bank erosion is controlled by climate, soil type, topography, vegetation, and the stream flow characteristics (Gray and Leiser, 1982, p. 12). The mechanisms that cause surface erosion can be

separated into those caused by rainfall and those due to fluvial processes of the stream. Streambank erosion and failure processes are discussed in Section 3.4.3.2.

5.3 COMPUTATIONAL METHODS FOR STABLE CHANNEL DESIGN

A preliminary channel design based on stability evaluation should be conducted early in project planning to screen out alternative designs that would present serious stability problems and to identify future needs. As planning progresses, successive evaluations with increasing detail may be required. This approach is essential to insure that the final channel design thoroughly addresses stability problems thus avoiding costly future channel maintenance efforts.

Channel design computations are based on a design discharge. The design discharge can be based on a computed hydrological event such as a 10-year storm event, or it can be based on the channel forming discharge which is responsible for shaping channel morphology. This section presents both preliminary and detailed design methods based on the channel forming discharge. Section 5.3.1 presents a detailed description of how to compute the channel forming discharge. Sections 5.3.2 - 5.3.5 present preliminary design methods, while Sections 5.3.6 - 5.3.9 present more computationally intensive methods for a detailed design.

5.3.1 CHANNEL FORMING DISCHARGE

An alluvial river adjusts the bankfull shape and dimensions of its channel of the wide range of flows that transport boundary sediments. However, for stable channels that are in equilibrium, a single, dominant flow can be identified which would produce a channel with the same morphological characteristics as the natural sequence of events. The concept of a single prevailing water and sediment discharge to which the river width, depth, slope, hydraulic roughness, and planform are adjusted is an attractive simplification. The single discharge can be used to assess general trends between channel morphologic characteristics and a single discharge. The single prevailing discharge represents a spectrum of discharges and is referred to as the channel forming discharge. The best situation for design would be to have gaged-defined water stage and discharge, and sediment discharge relationship defined at each site. In channel stabilization projects, the channel will be unstable, and therefore, it is unlikely a gage will exist.

Appendix A is a thorough examination of the proper computation procedure for effective discharge.

5.3.2 SLOPE-DRAINAGE AREA CURVE

The slope-drainage area curve is an empirical regional stability relationship that defines the stable channel slope, or equilibrium slope, as a function of drainage. The relationship is developed by field assessment to determine stable channel reaches. The slope of the stable reaches are determined by field survey and the drainage area at each stable site is determined from topographic maps. This slope is used

when siting grade control structures in unstable reaches. An example relationship is shown in Figure 5.26. For example, the drainage area of a particular reach is 100 square miles. The stable channel slope for that reach from Figure 5.26 is approximately 0.0010. Therefore, the grade control structures will be designed for a slope of 0.0010.

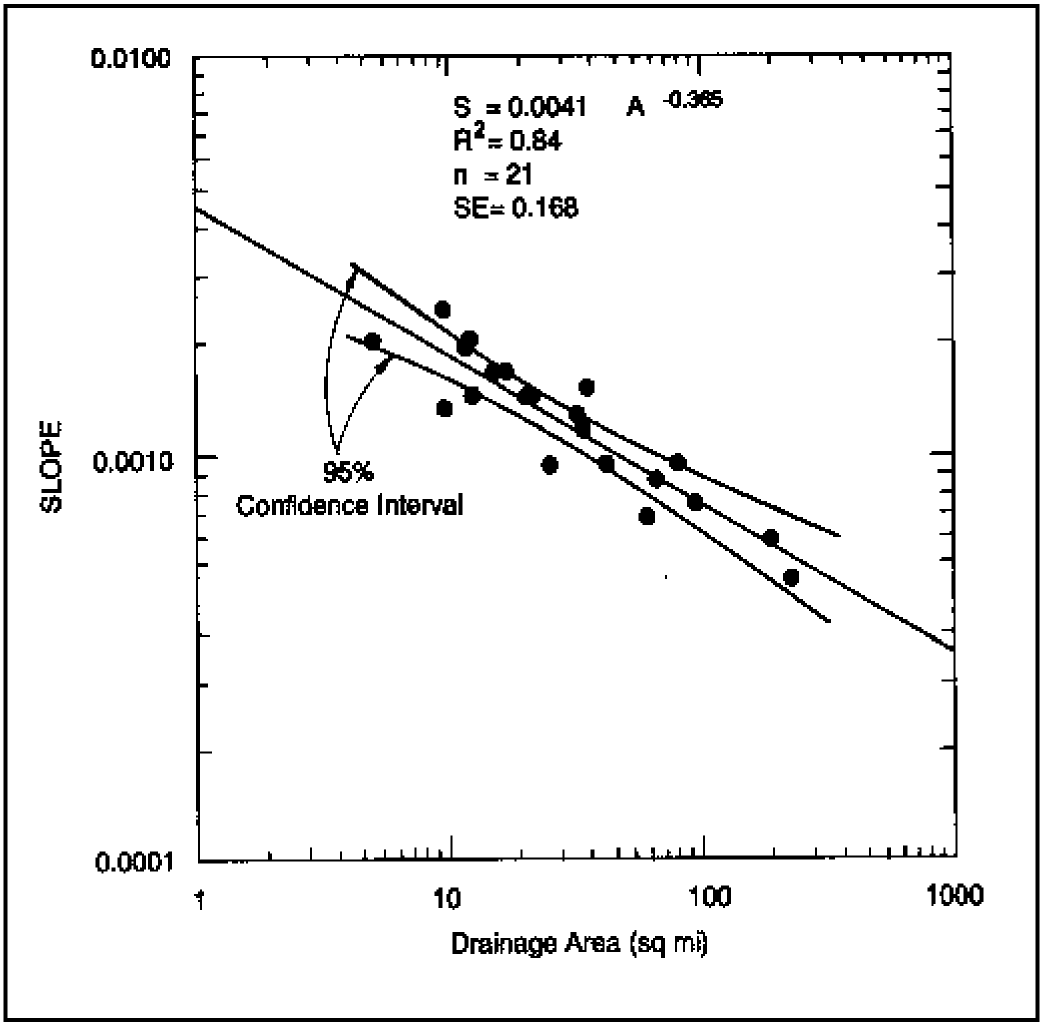


Figure 5.26 Equilibrium Channel Slope Versus Drainage Area for Hickahala Creek, Batupan Bogue and Hotopha Creek are Shown. The 95 Percent Confidence Intervals are Plotted (from USACE, 1990b).

The slope-drainage area curve is a valuable relationship for initial understanding of an unstable watershed. However, the relationship is empirical and extrapolation to other watersheds, beyond the range of size in the same watershed, or to different times is risky. Constant field verification is necessary for continued value.

5.3.3 MAXIMUM PERMISSIBLE VELOCITIES

The concept of maximum permissible velocity and the following tractive force design are closely linked. Both are based on the premise that excessive boundary shear stress in the channel will lead to erosion and stability problems. Two of the early references for permissible velocities are Etcheverry (1915) and Fortier and Scobey (1926). In 1915, Etcheverry provided the data found in Table 5.7 of maximum permissible velocities for irrigation canals (Fortier and Scobey, 1926). These velocities are for channels with no vegetative or structural protection.

Table 5.7 Maximum Mean Velocities Safe Against Erosion (Etcheverry, 1915)

Material	Mean velocity in ft/s
Very light pure sand of quicksand character	0.75 - 1.0
Very light loose sand	1.0 - 1.5
Coarse sand or light sandy soil	1.5 - 2.0
Average sandy soil	2.0 - 2.5
Sandy loam	2.5 - 2.75
Average loam, alluvial soil, volcanic ash soil	2.75 - 3.0
Firm loam, clay loam	3.0 - 3.75
Stiff clay soil, ordinary gravel soil	4.0 - 5.0
Coarse gravel, cobbles, shingles	5.0 - 6.0
Conglomerates, cemented gravel, soft slate, tough hard-pan, soft sedimentary rock	6.0 - 8.0
Hard rock	10.0 - 15.0
Concrete	15.0 - 20.0

In 1926, Fortier and Scobey presented a channel design method based on maximum permissible velocities for uniform flow. An earthen channel is considered stable if the mean velocity of the channel is less than the maximum permissible velocity for the channel. Their work is compiled based on a questionnaire given to a number of irrigation engineers whose experience qualified them to form authoritative estimates of the maximum mean velocities allowable in canals of various materials. The results of the questionnaire are given in Table 5.8. The USDA (1977) compiled data from Fortier and Scobey (1926), Lane (1953a,b), and the Union Soviet Socialist Republic (USSR, 1936) into a set of design charts. These charts are accompanied by a design procedure found in Technical Release No. 25, which is presented in the following figures and paragraphs.

Allowable Velocity Design Procedure (USDA, 1977)

1. Determine the hydraulics of the system. This includes hydrologic determinations as well as the stage-discharge relationships for the channel considered.

Table 5.8 Permissible Canal Velocities (Fortier and Scobey, 1926)

	Mean velocity, after aging of canals with flow depths # 3 ft					
Original material excavated for canals	Clear water, no detritus		Water transporting colloidal silts		Water transporting noncolloidal silts, sands, gravels or rock fragments	
	(ft/sec)	(m/sec)	(ft/sec)	(m/sec)	(ft/sec)	(m/sec)
1. Fine sand (noncolloidal)	1.5	0.46	2.5	0.76	1.5	0.46
2. Sandy loam (noncolloidal)	1.75	0.53	2.5	0.76	2.0	0.61
3. Silt loam (noncolloidal)	2.0	0.61	3.0	0.91	2.0	0.61
4. Alluvial silt (noncolloidal)	2.0	0.61	3.5	1.07	2.0	0.61
5. Ordinary firm loam	2.5	0.76	3.5	1.07	2.25	0.69
6. Volcanic ash	2.5	0.76	3.5	1.07	2.0	0.61
7. Fine gravel	2.5	0.76	5.0	1.52	3.75	1.14
8. Stiff clay	3.75	1.14	5.0	1.52	3.0	0.91
9. Graded, loam to cobbles (noncolloidal)	3.75	1.14	5.0	1.52	5.0	1.52
10. Alluvial silt (colloidal)	3.75	1.14	5.0	1.52	3.0	0.91
11. Graded, silt to cobbles (colloidal)	4.0	1.22	5.5	1.68	5.0	1.52
12. Coarse gravel (noncolloidal)	4.0	1.22	6.0	1.83	6.5	1.98
13. Cobbles and shingles	5.0	1.52	5.5	1.68	6.5	1.98
14. Shales and hard pans	6.0	1.83	6.0	1.83	5.0	1.52

2. Determine the soil properties of the bed and banks of the design reach and of the channel upstream.

3. Determine sediment yield for the reach and compute sediment concentration for design flow.

4. Check to see if the allowable velocity procedure is applicable using the Channel Evaluation Procedural Guide, Figure 5.27.

5. Determine the basic channel velocities from Figure 5.28a and multiply them by the appropriate correction factors as found in Figure 5.28b. Compare the design velocities with the allowable velocities determined from Figures 5.28a and 5.28b.

6. If the allowable velocities are greater than the design velocities, the design is satisfactory. Otherwise, if the allowable velocities are less than design velocities, it may be necessary to consider a mobile boundary condition and evaluate the channel using appropriate sediment transport theory and programs.

5.3.4 TRACTIVE FORCE DESIGN

Lane (1953a,b) developed an analytical design approach for shear distribution in trapezoidal channels. The tractive force, or shear force, is the force which the water exerts on the wetted perimeter of a channel due to the motion of the water.

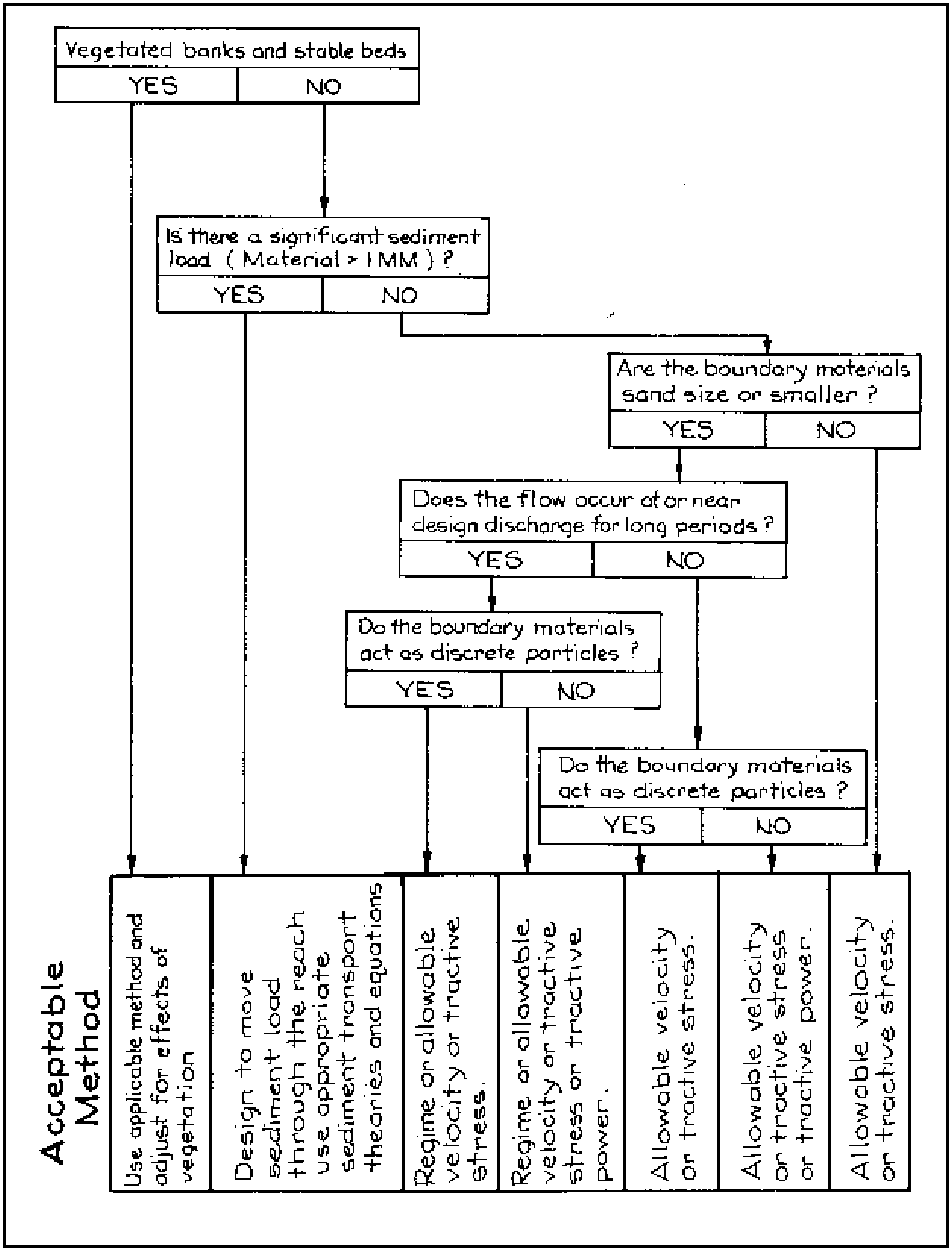
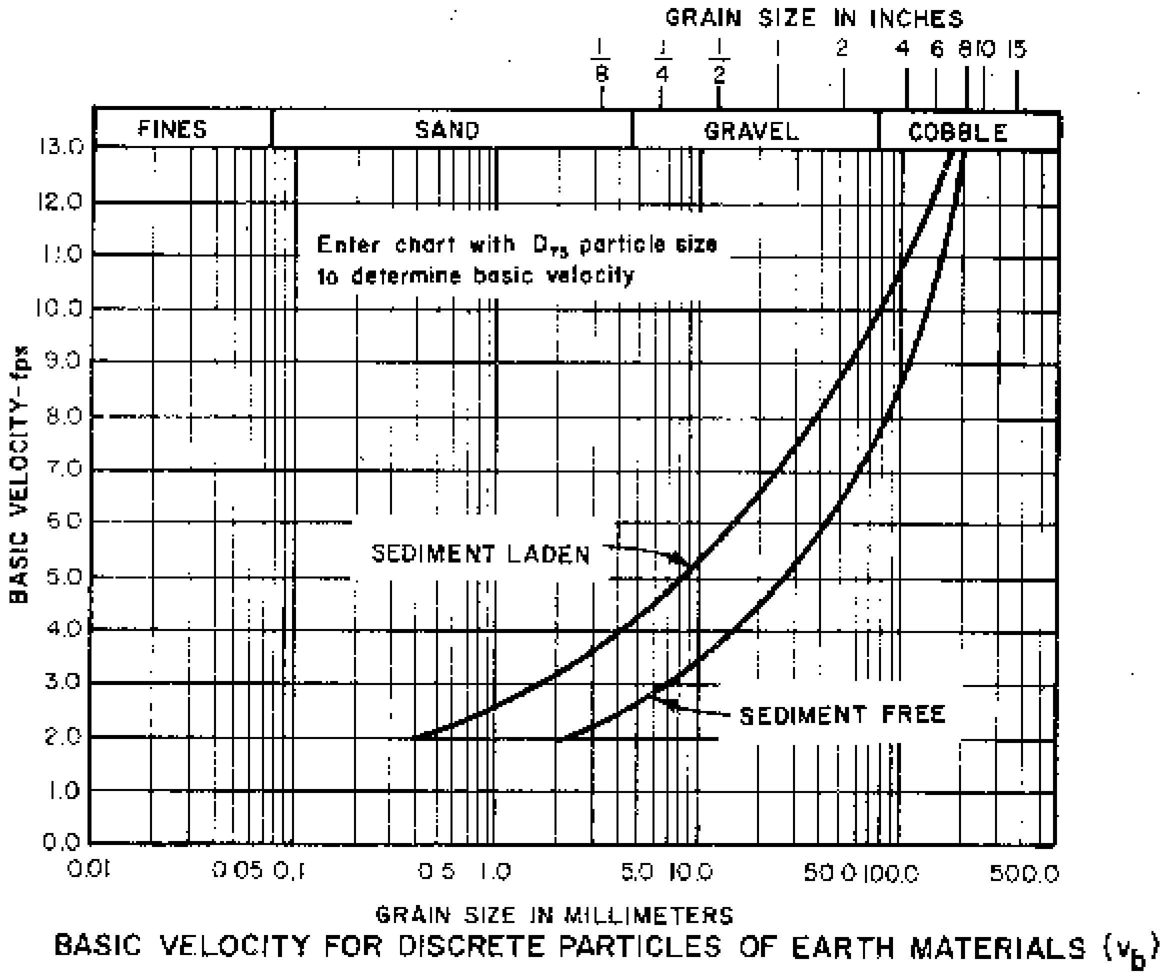


Figure 5.27 Channel Evaluation Procedural Guide (from USDA, 1977)



ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS	
CHANNEL BOUNDARY MATERIALS	ALLOWABLE VELOCITY
DISCRETE PARTICLES	
Sediment Laden Flow	
$D_{75} \geq 0.4$ mm	Basic velocity chart value x D x A x B
$D_{75} \leq 0.4$ mm	2.0 fps
Sediment Free Flow	
$D_{75} \geq 2.0$ mm	Basic velocity chart value x D x A x B
$D_{75} \leq 2.0$ mm	2.0 fps
COHERENT EARTH MATERIALS	
PI $\geq$ 10	Basic velocity chart value x D x A x F x C_e
PI $\leq$ 10	2.0 fps

Figure 5.28a Allowable Velocities for Unprotected Earth Channels (from USDA, 1977)

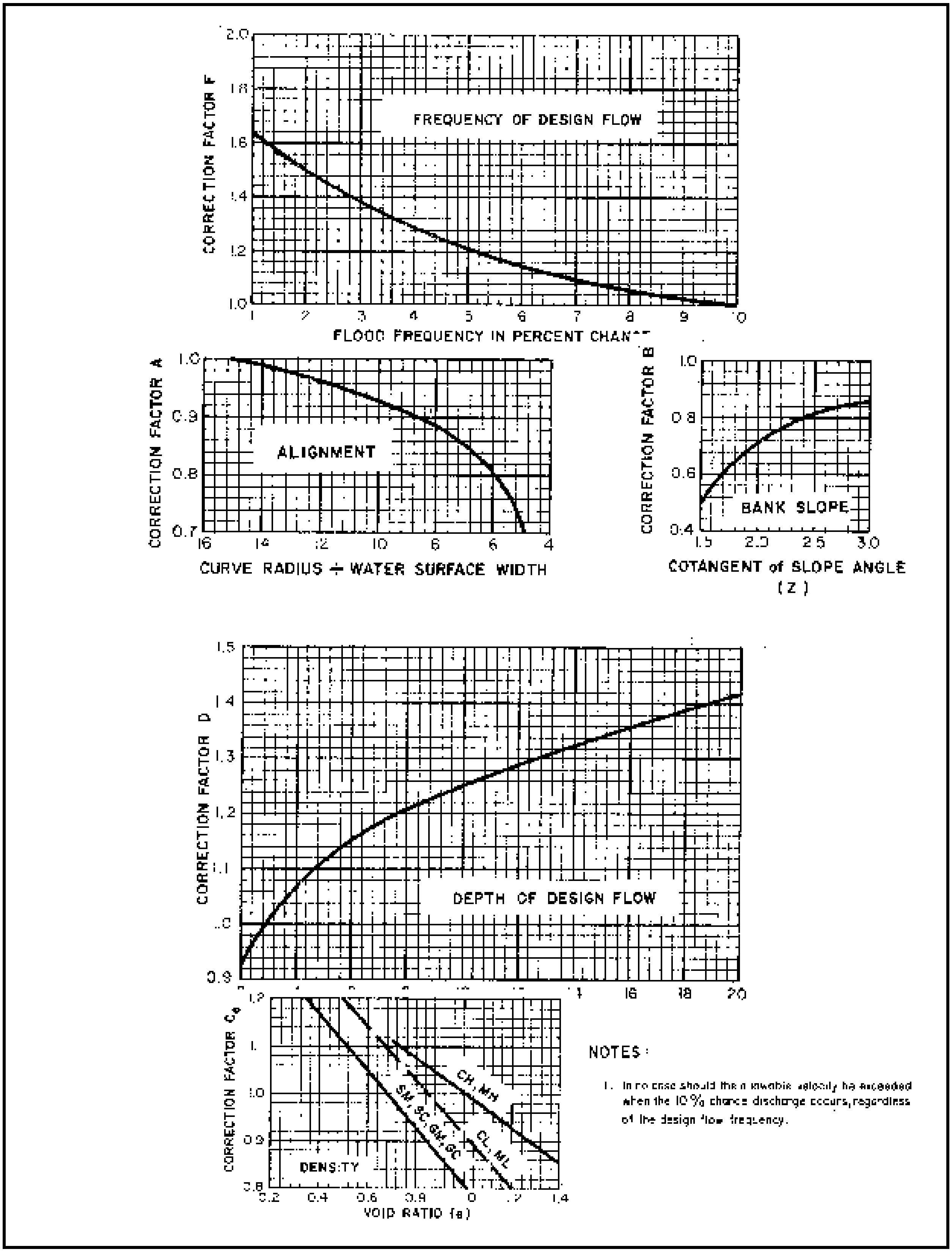


Figure 5.28b (cont.) Allowable Velocities for Unprotected Earth Channels (from USDA, 1977)

It is not the force on a single particle but rather the force exerted over a certain area of the bed or banks. It is equal to and in the opposite direction from the force which the bed exerts on the flowing water. It is the force which is the component, in the direction of flow, of the weight of water (Lane, 1953b). The weight force is equal to $\gamma A L S_f$, where: γ is the specific weight of water, A is the cross-sectional area, L is the length of the channel reach, R is the hydraulic radius, P is the wetted perimeter, and S_f is the slope of the energy grade line. The average value of the tractive force per unit wetted perimeter or the unit tractive force is given by the following equation (Simons and Sentürk, 1992):

$$\tau_o = \frac{\gamma A L S_f}{P L} = \gamma R S_f \tag{5.3}$$

Lane shows that in most canals, similar to those used for irrigation, the tractive force near the middle of the channel invert closely approaches $\gamma\ D\ S_o$, where: D is the hydraulic depth of the channel, and S_o is the slope of the bed assuming uniform flow. His results indicate that for trapezoidal channels the maximum shear on the sides is approximately 0.75 $\gamma\ D\ S_o$ as illustrated in Figure 5.29. Figure 5.30 shows the maximum shear for the sides and the bottom of trapezoidal sections in a graphical format. Lane found the side slopes of a channel posed limitations on the maximum allowable shear force in the channel. He analyzed the shear and gravity forces acting on a sediment particle on the canal side slope to quantify these effects. K is defined as the ratio of the tractive force necessary to start motion on the sloping side of a canal, to that required to start motion for the same material on a level surface as the following (Lane, 1953b):

$$K = \cos\phi \sqrt{1 - \frac{\tan^2\phi}{\tan^2\theta}} \tag{5.4}$$

where: ϕ = the angle with the horizontal of the side slope of the canal; and
θ = the angle with the horizontal of repose of the material.

Lane presents a graphical representation of this equation as seen in Figure 5.31. Simons (1957) provides a detailed process for Lane's tractive stress method.

Tractive Force Design Procedure (Simons, 1957)

1. Knowing Q and d_{75}, assume a shape.

2. Calculate a width to depth ratio, B/D, based on assumed shape. Enter Figure 5.30 with this arbitrary value and determine the magnitude of C in the equation $\tau_c = C\ \gamma\ D\ S_o$.

   where: τ_c = critical tractive force;
   C = friction coefficient;
   γ = specific weight of water;
   D = hydraulic depth of channel; and
   S_o = bed slope.

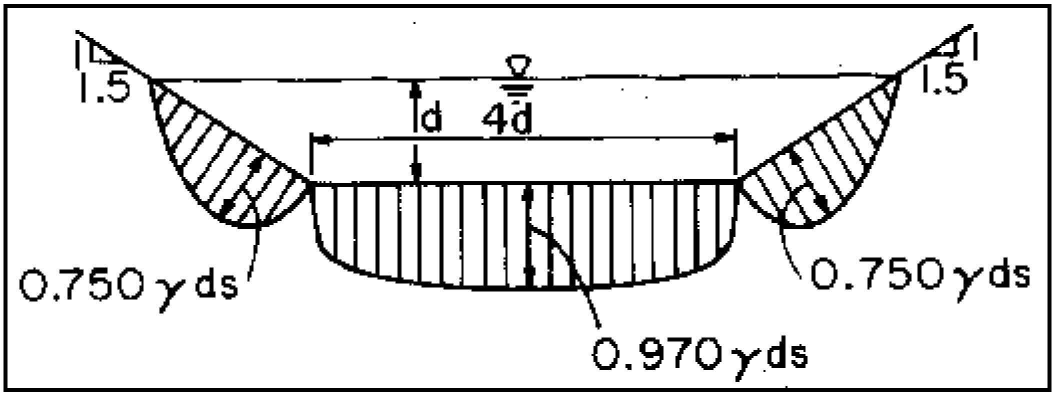


Figure 5.29 Maximum Unit Tractive Force Versus b/d (from Simons and Sentürk, 1992), b is the Bottom Width and d is the Depth

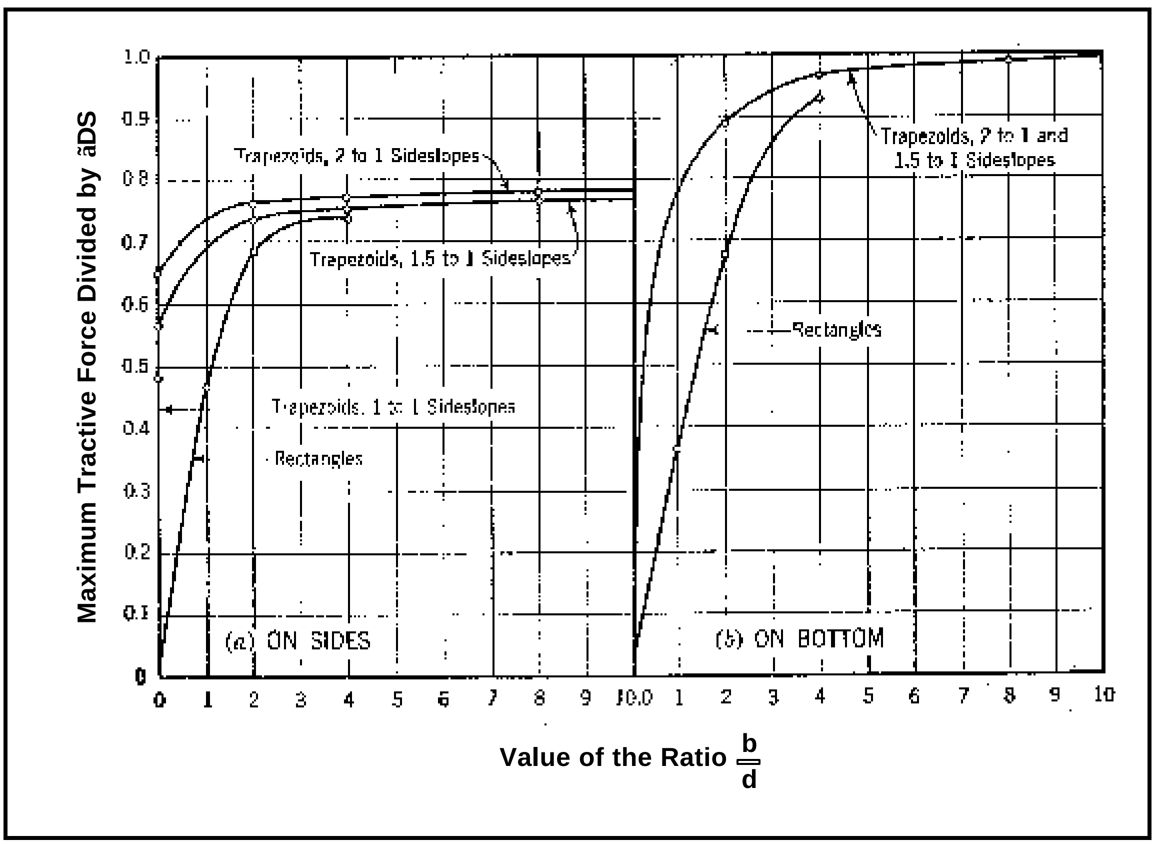


Figure 5.30 Maximum Tractive Forces in a Channel (from Lane, 1953b)

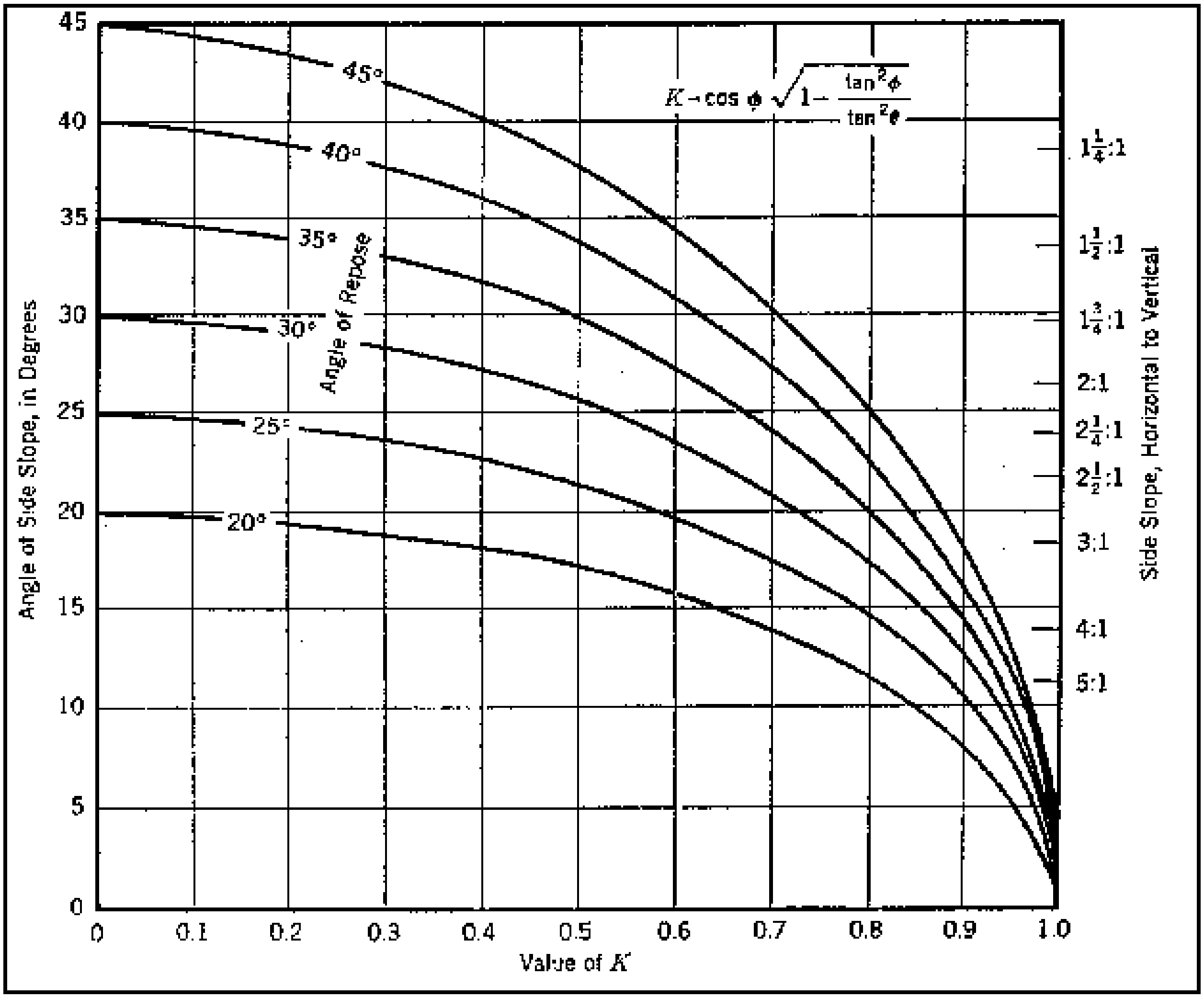


Figure 5.31 Relationship Between Side Slope and K (from Lane, 1953b)

3. Determine the value of ô, tractive force, corresponding to the d_{75} from Figure 5.32.

4. Based on bed conditions estimate the maximum permissible longitudinal slope by equating the value of ô taken from Figure 5.32 to C ã D S_o and solve for S_o in the form:

$$S_o \; ' \; \frac{J}{C \; (\; D} \tag{5.5}$$

The influence of the stability of the canal sides on channel slope S_o must now be checked. Usually, the side material cannot resist as great a tractive force as the bed because of the additional effect of gravity.

5. Knowing size and shape of material, enter Figure 5.33 and estimate the angle of repose.

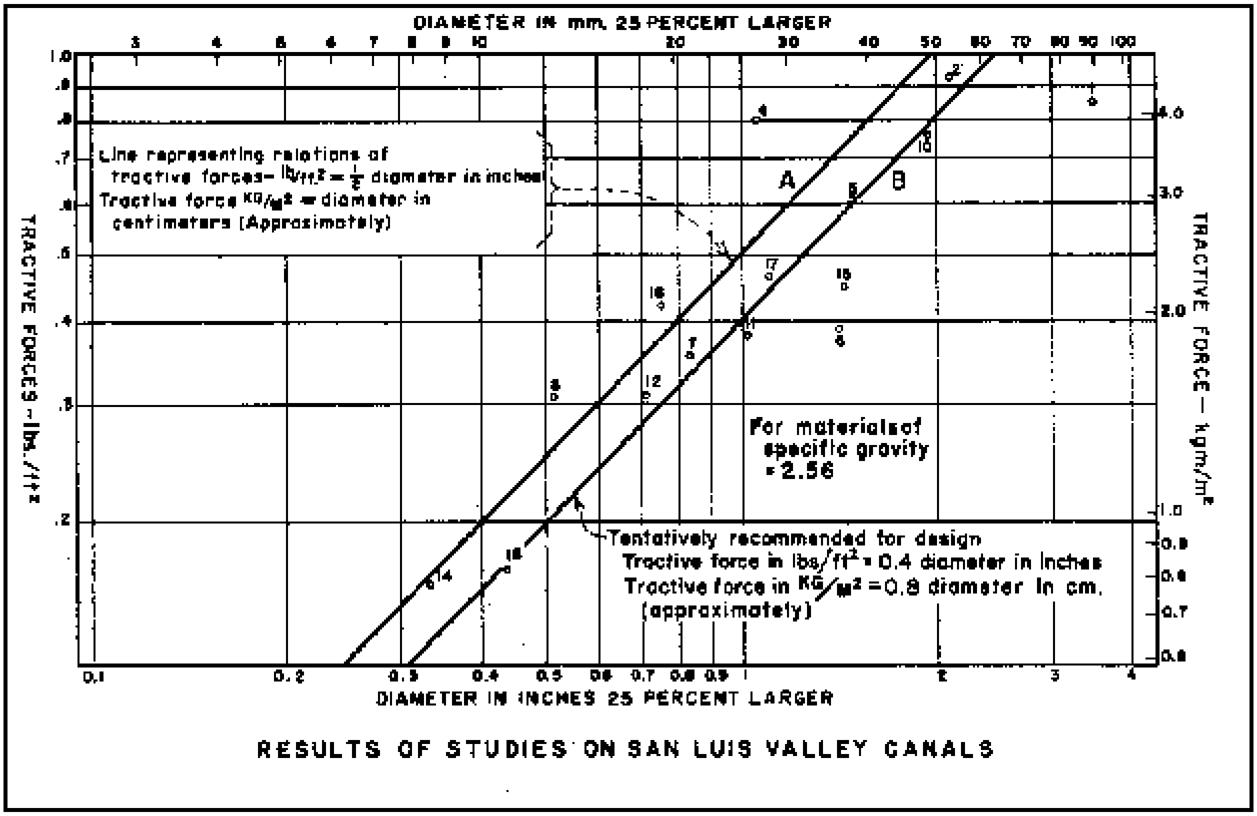


Figure 5.32 Variation of Tractive Force ô With Bed Material d (from Lane, 1953a)

6. Evaluate K from Figure 5.31. Knowing K and the critical tractive force acting on the bed, the tractive force on the sides can be computed.

7. Enter Figure 5.30 and determine the maximum tractive force in terms of ã D S_o acting on the sides of the canal. That is, determine C in the expression ô = C ã D S_o.

8. Equate ô from step 6 to C ã D S_o and knowing C, the slope S_o can be evaluated.

9. Compare the slope based on bed stability, step 4, with slope based on side stability, step 8, whichever is smaller governs.

10. Check the capacity of the canal using the established slope and assumed shape. If the capacity is incorrect, assume a new shape and repeat the above procedure. This process continues until a satisfactory design results.

The following limitations of the maximum allowable velocity and tractive force design procedures are as follows (USACE, EM 1110-2-1418, 1994):

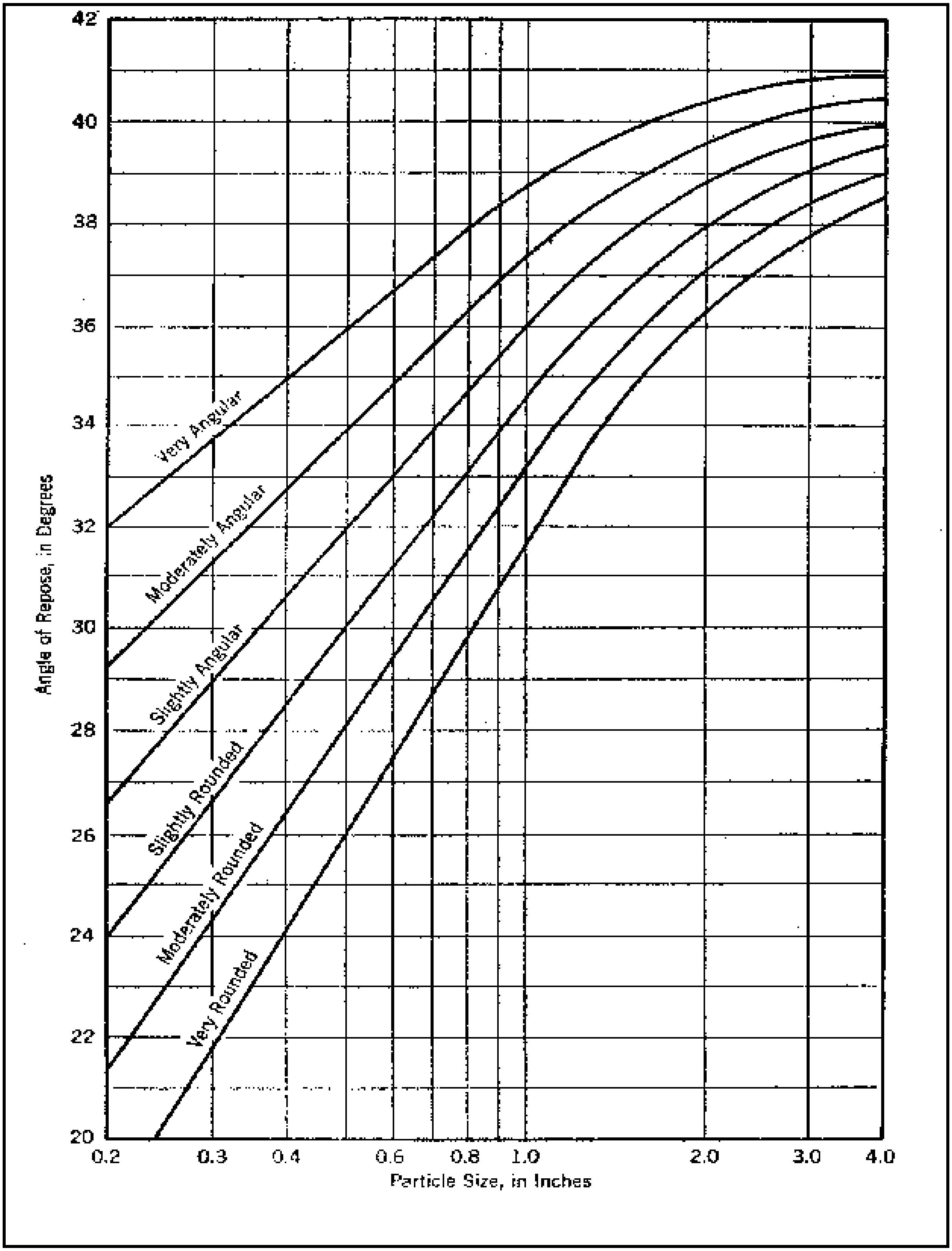


Figure 5.33 Angle of Repose of Noncohesive Material (from Lane, 1953b)

1. For channels with substantial inflows of bed materials, a minimum velocity or shear stress to avoid deposition may be as important as a maximum to avoid erosion. Such as value cannot be determined using allowable data for minimal erosion.

2. In bends and meandering channels, bank erosion and migration may occur even if average velocities and boundary shear stresses are well below allowable values.

3. An allowable velocity or shear stress will not in itself define a complete channel design, because it can be satisfied by a wide range of width, depth, and slope combinations.

4. The shear stress computations apply to a uniform flow over a flat bed. In sand channels the bed is normally covered with bed forms such as ripples or dunes, therefore shear stresses required for significant erosion may be much greater than that indicated in the computations.

5.3.5 REGIME THEORY CHANNEL DESIGN

Regime theory is not a theory in the strict sense of the term, for it does not incorporate physical explanations for findings. The essence of the system lies in the development of convenient and simple empirical equations from field data collected from rivers and from successfully operating artificial canals (Henderson, 1966). In 1895, Kennedy (Lacey, 1931) developed the first well known regime equation in India on the Upper Bari Doab Canal. He used the silt of the Upper Bari Doab Canal as a standard of reference to quantify sedimentation on canal systems. Many relationships have been developed from the Indian canals.

In the United States, Simons and Albertson (1963) continued regime development by combining data from canal studies in India (Punjab and Sind) and the United States (Imperial Valley, San Luis Valley, and canals in Wyoming, Colorado, and Nebraska). Their motive for additional development of regime analysis is the inadequacy of previous regime methods. The three primary inadequacies are (Simons and Albertson, 1963):

1. the regime equations have not been developed based on the wide variety of conditions encountered in practice;

2. the regime equations fail to recognize the important influence of sediment transport on design; and

3. the regime equations involve factors that require a knowledge of the conditions upon which the formulas are based if to be applied successfully.

Their data are separated into three groups based on the composition of streambed and streambanks. This eliminates the need for computing bed, bank or silt factors needed for previous types of equations (Watson and Abt, 1991). Simons and Albertson (1963) equations are referred to as the Modified Regime Equations and are presented in Table 5.9.

Table 5.9 Simons and Albertson (1963) Modified Regime Equations

	Sand Bed and Sand Banks	Sand Bed and Cohesive Banks	Cohesive Bed and Cohesive Banks
$P = C_1\, Q^{0.512}$	3.3	2.51	2.12
$R = C_2\, Q^{0.361}$	0.37	0.43	0.51
$A = C_3\, Q^{0.873}$	1.22	1.08	1.08
$V = C_4\, (R^2\, S)^{1/3}$	13.9	16.1	16.0
$W/D = C_5\, Q^{0.151}$	6.5	4.3	3.0

Simons and Albertson (1963) explain the limitations of the Indian and their own regime equations. Simons and Albertson (1963) also provide guidance for designing with their equations:

1. Canals that are formed in coarse non-cohesive material of the type studied by the USBR (sediment transport < 500 ppm).

2. Canals that are formed in sandy material with sand beds and banks (sediment transport < 500 ppm).

3. Canals that are formed in sand beds and slightly cohesive to cohesive banks (good results when sediment transport < 500 ppm, qualitative results when sediment transport > 500 ppm).

4. Canals having cohesive beds and banks (sediment transport < 500 ppm).

The USACE (1994) provides guidance on channel design. Their recommendation is to use locally or regionally developed equations for channel design. However, when this is not possible, Figures 5.34, 5.35, and 5.36 can be used to provide rough estimates for top width, depth, and slope of a channel given the channel-forming discharge and bed material. Limitations associated with the charts are provided in the following paragraphs.

USACE Regime Chart Limitations

1. Where possible, reach-averaged data for existing channels should be plotted and compared with the indications of the charts, using bankfull discharge as the channel-forming. If bankfull discharge is not determinable, a 2-year recurrence discharge can be used as the channel forming. This comparison can indicate how compatible the stream system is with the assumptions of the charts. The trends of the charts can then be used to estimate changes appropriate for modifications due to increased in-channel flows.

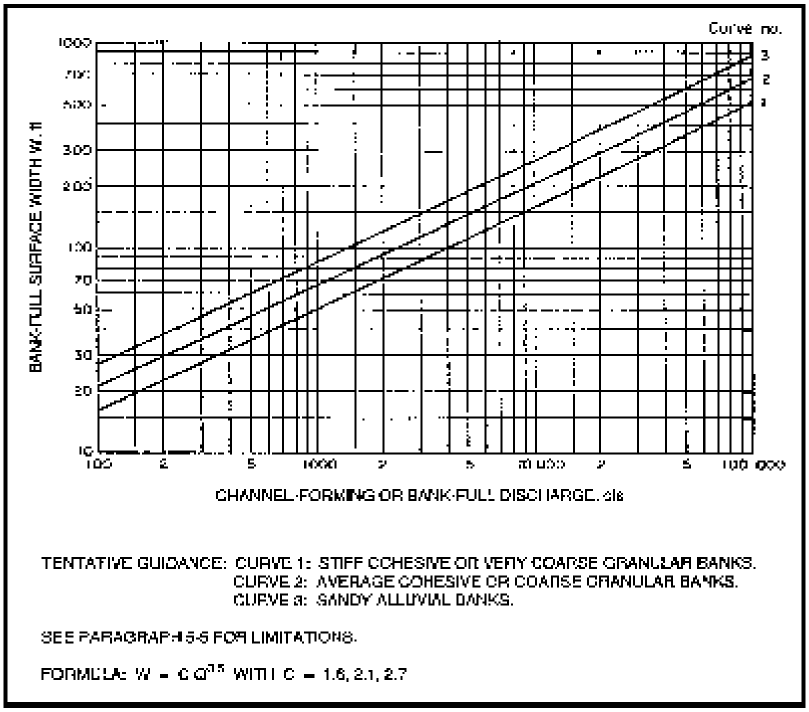


Figure 5.34 Top Width as Function of Discharge (USACE, 1994)

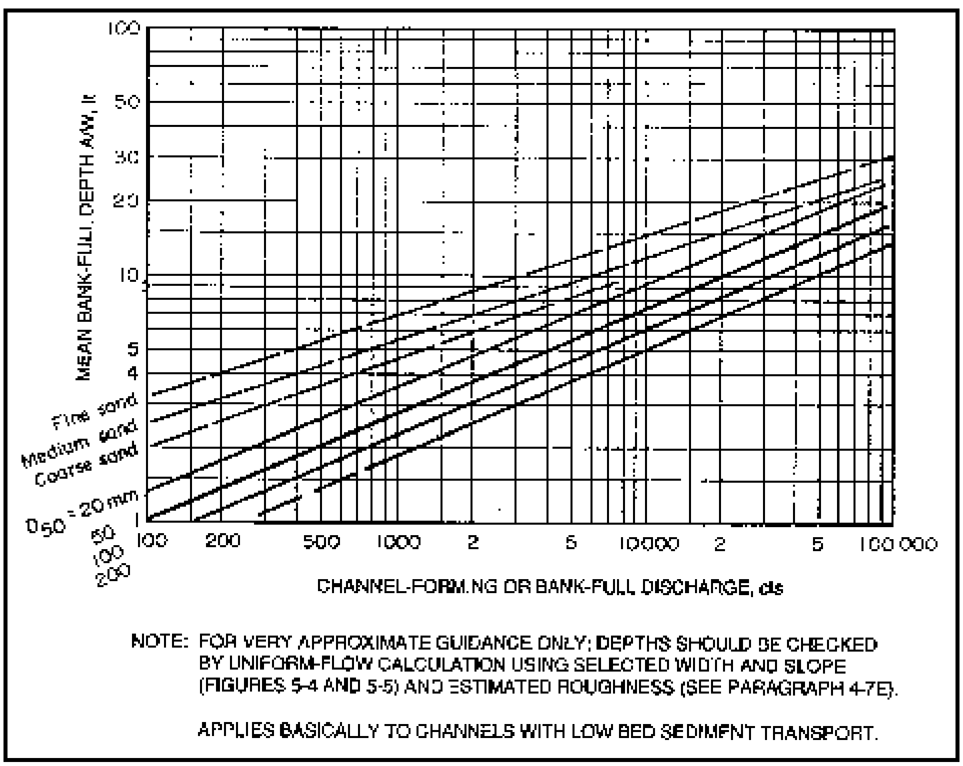


Figure 5.35 Depth as Function of Discharge (from USACE, 1994)

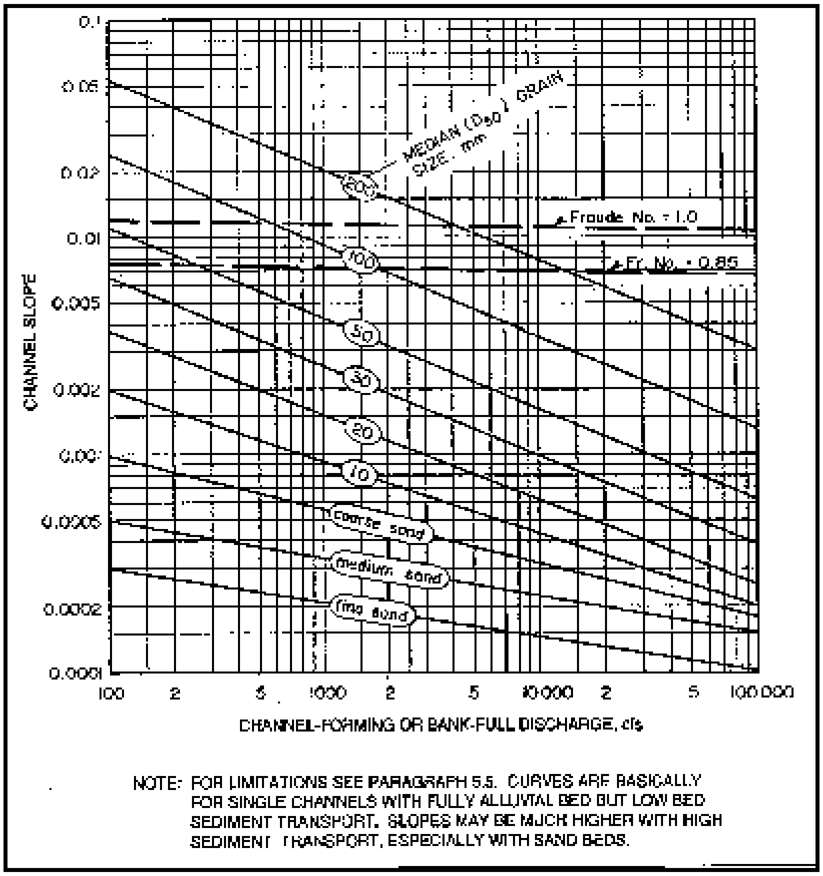


Figure 5.36 Slope as Function of Discharge (USACE, 1994)

2. The charts are more compatible with single-channel sand or gravel systems with relatively low bed material transport. A multichannel system will tend to have greater overall widths and slopes but smaller depths. However, individual branches may fit the curves reasonably in relation to their partial bank-full discharges.

3. If bed material transport is high, the slopes indicated in Figure 5.36 may be too low and the depths in Figure 5.35 may be too high. This is especially true for channels with sand beds and of ephemeral channels where much of the flow occurs as flash floods with very high sediment transport. In perennial-flow gravel rivers with single channels, slopes are unlikely to be more than three times greater than those indicated by Figure 5.36. Width is fairly insensitive to bed material transport unless the stream is multichanneled or braided. If bed material transport is high, it is preferable to use a sediment budget analysis. This is when field observations and checks of velocity, shear stress, or hydraulic geometry indicate a substantial degree of actual or potential bed instability and sediment transport.

4. Actively aggrading and degrading channels can go through a complex cycle of response, which may exhibit large departures from normal hydraulic geometry relationships. For example, a channel in the earlier stages of active incision may be abnormally narrow.

5. The use of all three charts does not permit explicit selection of roughness and allowable velocity or shear stress. An alternative hybrid approach involves determining channel properties using three relationships: a) the top width-discharge relationship of Figure 5.34; b) the Manning formula with roughness estimate based on guidelines or experience; and c) an allowable velocity or shear stress.

5.3.6 HYDRAULIC DESIGN PACKAGE FOR CHANNELS (SAM)

Thomas *et al*. (1994) developed the Hydraulic Design Package for Channels (SAM). SAM is a computer program available through the USACEWES. The program was developed to provide the designer with a tool to assist in sediment transport calculations and the design of stable flood control channels. SAM is organized into three major modules: 1) sediment transport calculations, 2) sediment yield calculations, and 3) hydraulic calculations. Additional information on the capabilities of SAM and availability can be found on the website **http://chl.wes.army.mil/ software/sam/**.

5.3.6.1 Sediment Transport Calculations

The sediment transport module of SAM computes sediment transport rate as a function of known hydraulic parameters. Currently there are 19 sediment transport equations incorporated into SAM. Typically sediment transport is calculated based on the probabilistic distribution of multiple grain size classes in the bed material. However, some of the equations calculate sediment transport using a single characteristic grain size (D_{50}). The sediment transport equations included in SAM are listed:

- Schoklitsch
- Meyer-Peter and Müller
- Meyer-Peter and Müller, D_{50}
- Parker
- Einstein bedload
- Einstein total load
- Englund-Hansen
- Toffaleti
- Toffaleti-Schoklitsch
- Toffaleti-Meyer-Peter and Müller
- Yang
- Yang, D_{50}
- Acker-White
- Acker-White, D_{50}

- Colby
- Brownlie, D_{50}
- Laursen (Madden)
- Laursen (Copeland)
- Profitt (Sutherland)

The sediment transport module can be used to develop sediment rating curves from any of the 19 equations. A sediment rating curve yields the sediment transport rate as a function of discharge rate. An example of a sediment rating curve is given in Figure 5.37. The selection of an appropriate sediment transport equation should be based on the range of particle sizes in the bed material and the flow conditions being investigated.

5.3.6.2 Sediment Yield Calculations

Sediment yield is the weight of sediment passing a cross-section during a specified period of time (Thomas *et al*., 1994). Typically sediment yield is evaluated on an annual bases, but calculations can be performed for a single event. SAM offers two options for computing sediment yield: the flow duration curve method and the flow hydrograph method.

The flow duration curve method integrates a flow duration curve with a sediment transport rating curve to evaluate the total sediment passing the basin outlet. A flow duration curve is a cumulative distribution function which presents the percentage of time during an average year that a given discharge is equaled or exceeded. An example flow duration curve is given in Figure 5.38. Sediment transport rating curves are described in the previous section. SAM uses a log-linear interpolation of the discharge versus exceedence probability flow duration curve. A log-log interpolation of the sediment transport rating curve is used. The flow hydrograph method integrates a hydrograph with a sediment rating curve to evaluate the sediment yield for a given event. A hydrograph is a plot of discharge versus time, Figure 5.39. This method is used to evaluate the sediment yield for a given event for which the hydrograph is known.

5.3.6.3 Hydraulic Calculations

The hydraulics calculations module in SAM evaluates channel dimensions in both fixed and mobile bed boundaries. The module calculates channel dimensions by solving the Manning equation, calculating stable channel dimensions using Copeland's method, and sizing riprap for channel stability.

5.3.6.4 Governing Equations for Stable Channel Design Procedure

Copeland's method for stable channel design is an analytical technique that calculates channel dimensions by simultaneously solving equations which govern water and sediment continuity. The method uses Brownlie (1981) for flow resistance and sediment transport equations.

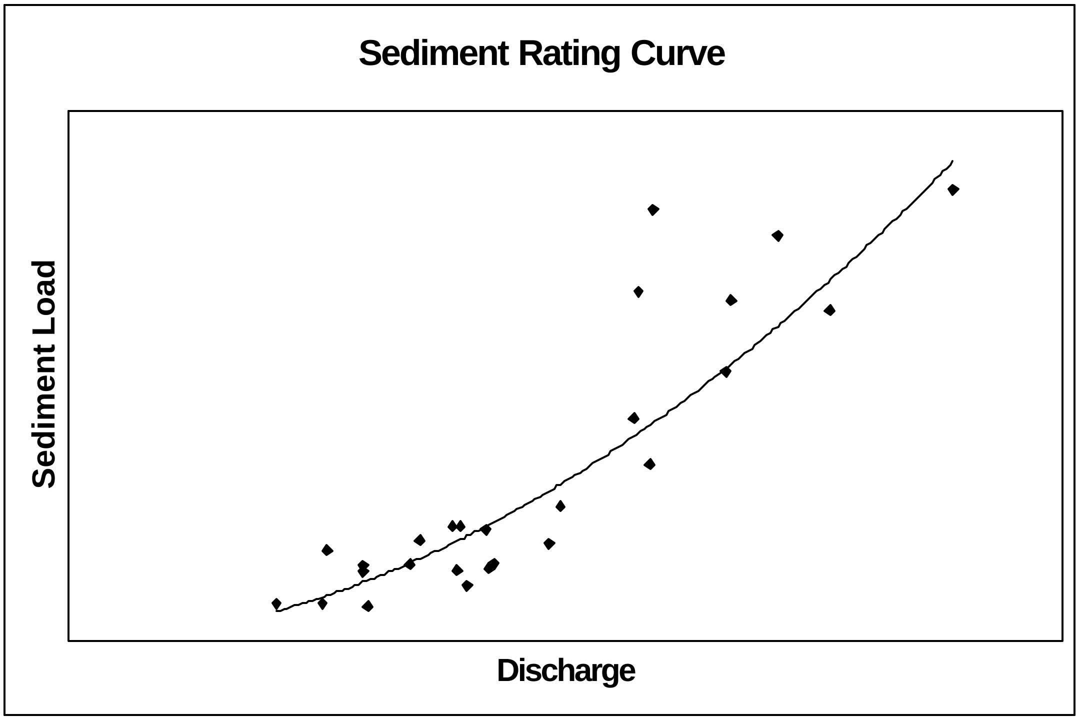


Figure 5.37 Example of a Sediment Rating Curve

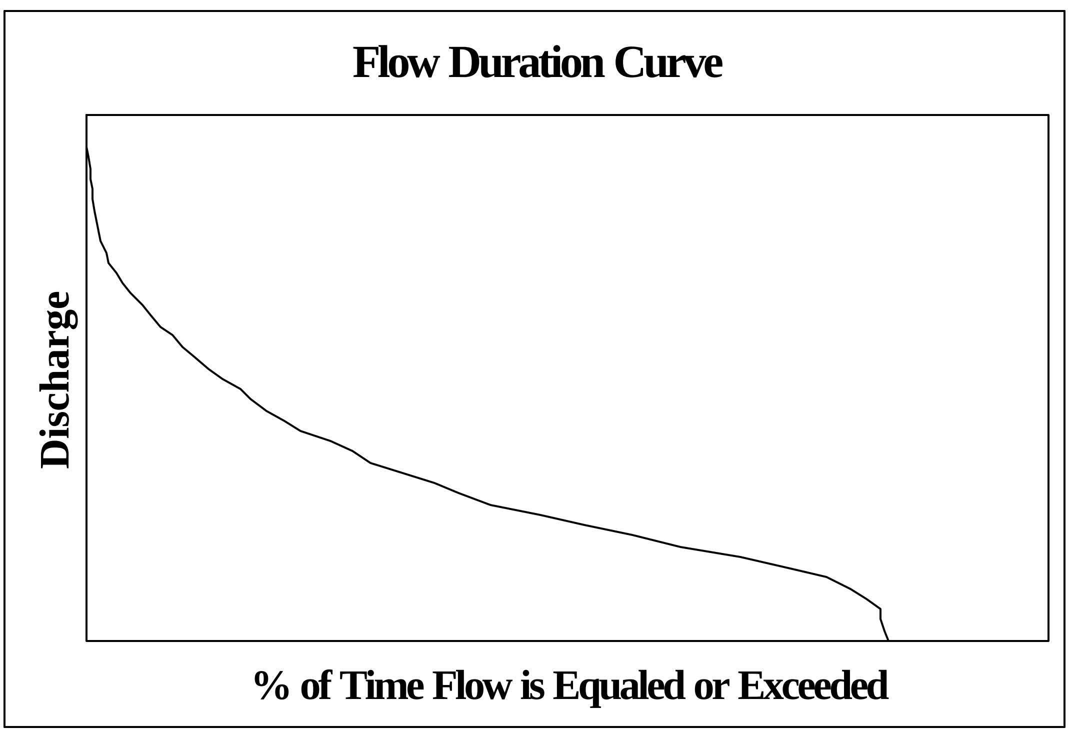


Figure 5.38 Example of a Flow Duration Curve

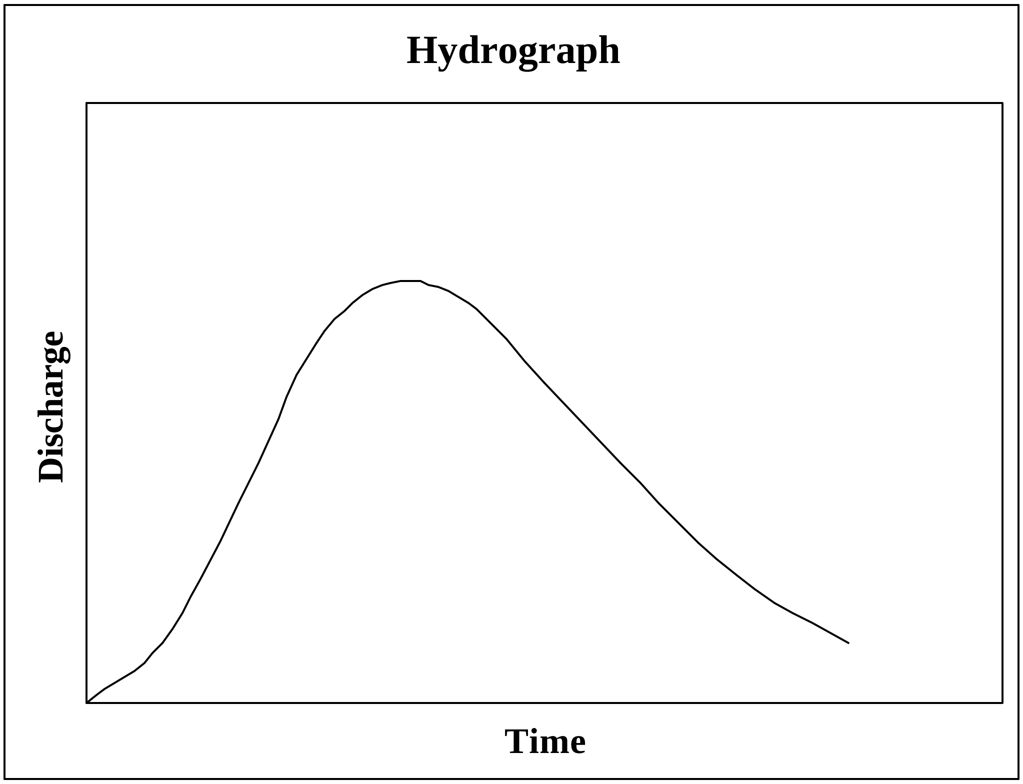


Figure 5.39 Example of Hydrograph

These equations are for sand bed rivers, based on approximately 1,000 records from 31 sets of laboratory and field data. Brownlie's equations for predicting flow resistance are used to compute the bed roughness in SAM. The flow resistance equations are based on four dimensionless quantities: grain Froude number (F_g); ratio of the median grain size to the laminar sublayer (D_{50}/ä); the bed slope (S); and the geometric bed material gradation coefficient (ó$_g$). These quantities account for both bedform and grain roughness in a channel cross section. However, the gradation coefficient, ó$_g$, is reported by Brownlie to have a small effect on his analysis.

Bedform roughness is the roughness that is the result of bedforms such as ripples or dunes. These bedforms occur in the lower regime when the flow is generally subcritical ($Fr < 1$) (Julien, 1995). Form roughness varies with the flow rate in the channel. Therefore, a small change in the discharge, may have a considerable impact on the computed stable channel dimensions.

The grain roughness is the roughness associated with the size of the sediment particles on the bed. This type of roughness typically dominates in the upper flow regime (Julien, 1995). However, upper regime can occur at subcritical and supercritical flow as shown by Athaullah (1968), Figure 5.40.

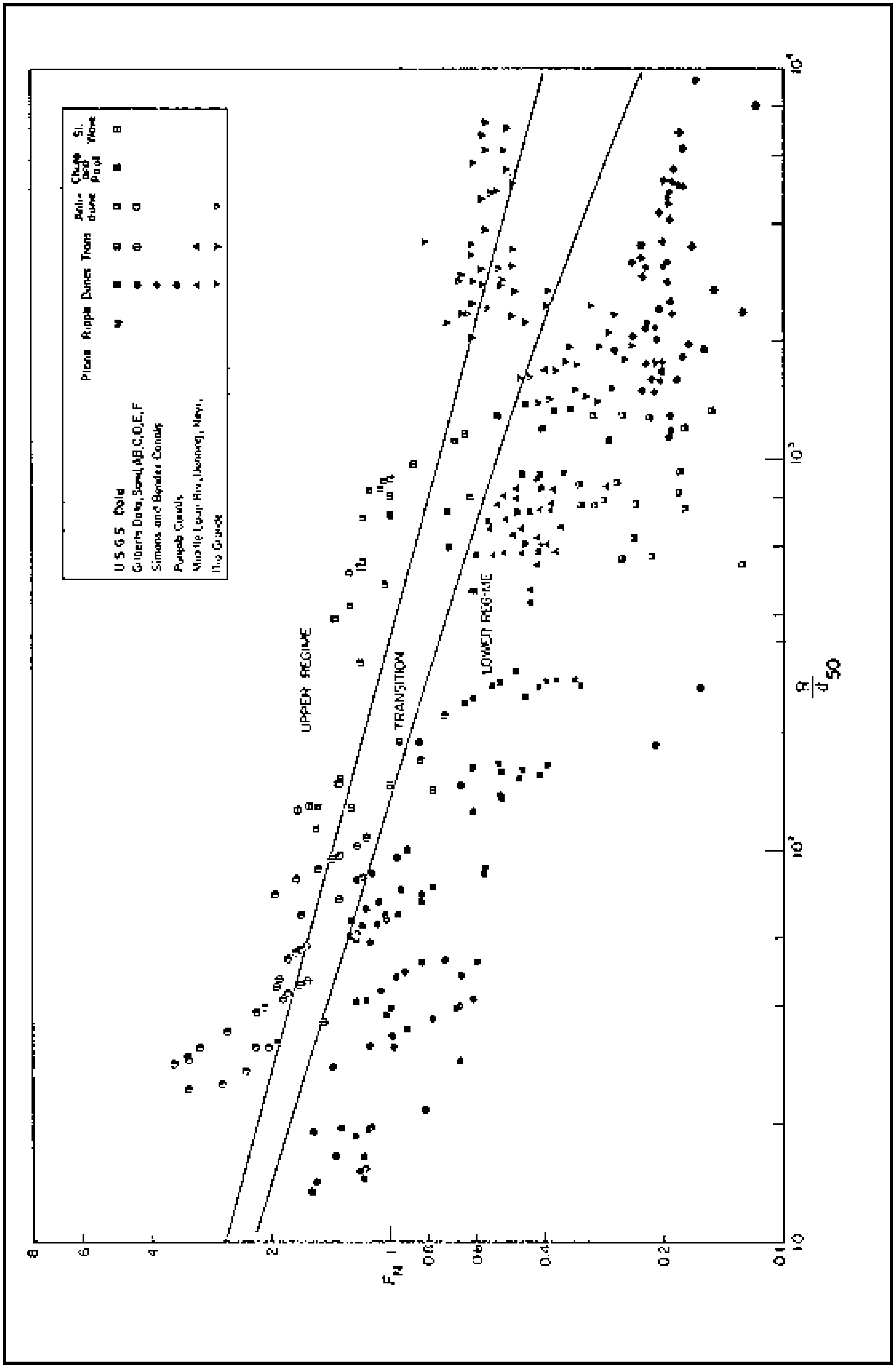


Figure 5.40 Froude Number F_N Versus R/D_{50} Criterion

F_g is the grain Froude number representing the square root of the ratio of drag forces on a particle to the particle weight. Brownlie defines the grain Froude number by the following equation (Brownlie, 1981):

$$F_g = \frac{\sqrt{\rho}\, V}{\sqrt{(\rho_s - \rho)\, g D_{50}}} \tag{5.6}$$

where: ρ = density of water;
ρ_s = density of sediment particles;
V = depth average velocity;
D_{50} = median grain size of the particles; and
g = acceleration due to gravity.

D_{50}/δ is the ratio of the mean grain size of the particles to the thickness of the laminar sublayer. It is defined by:

$$\frac{D_{50}}{\delta} = \frac{u_* D_{50}}{11.6\,\nu} \tag{5.7}$$

where: δ = laminar sublayer thickness;
u_* = shear velocity; and
ν = kinematic viscosity of water.

Brownlie plotted the grain Froude number versus the slope for all upper and lower regime data to incorporate the slope, Figure 5.41. Brownlie reported that beyond a slope of $S = 0.006$, only the upper flow regime exists. For values lower than 0.006 an approximate dividing line in the data may be defined by $F_g = F_g'$. F_g' is computed from the following regression analysis as developed from Figure 5.41.

$$F_g' = 1.74\, S_f^{-0.33} \tag{5.8}$$

Brownlie plotted the F_g/F_g' versus D_{50}/δ for transitional data with slopes less than 0.006. Division of F_g by F_g' eliminates the bias found when plotting F_g against just the slope. This defines the transition between upper and lower regime as seen in Figure 5.42. The transition region is defined by Eqs. (5.7) and (5.8) for the lower and upper limits of the flow regimes.

For the lower limit of the upper flow regime, $D_{50}/\delta < 2$:

$$\log \frac{F_g}{F_g'} = -0.02469 + 0.1517 \log\left(\frac{D_{50}}{\delta}\right) + 0.8381\left[\log\left(\frac{D_{50}}{\delta}\right)\right]^2 \tag{5.9}$$

and log 1.25 for $D_{50}/\delta \geq 2$. For the upper limit of the lower flow regime, $D_{50}/\delta < 2$:

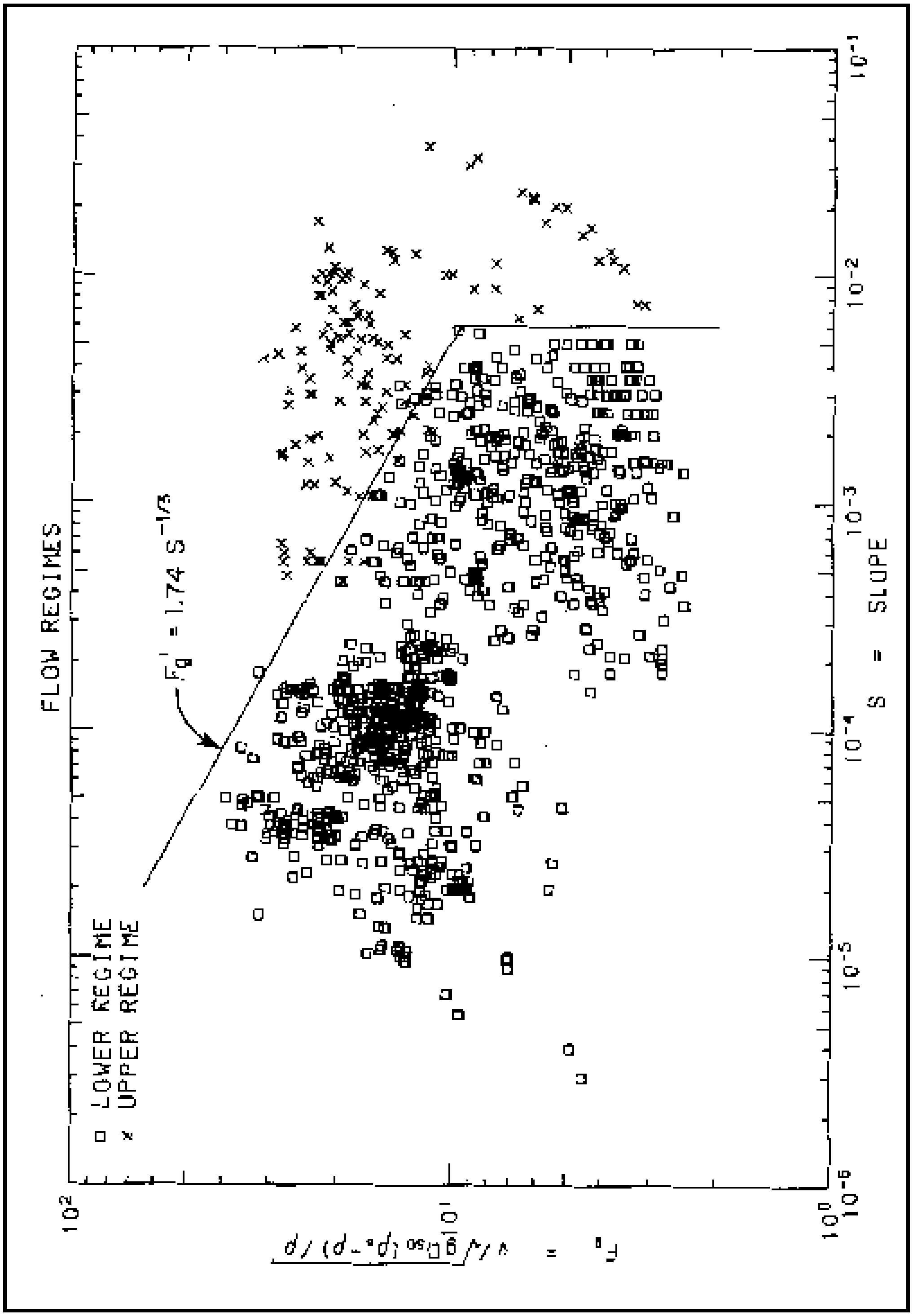


Figure 5.41 Determination of Flow Regimes - Grain Froude Number, F_g, Plotted Against Slope, S (from Brownlie, 1981)

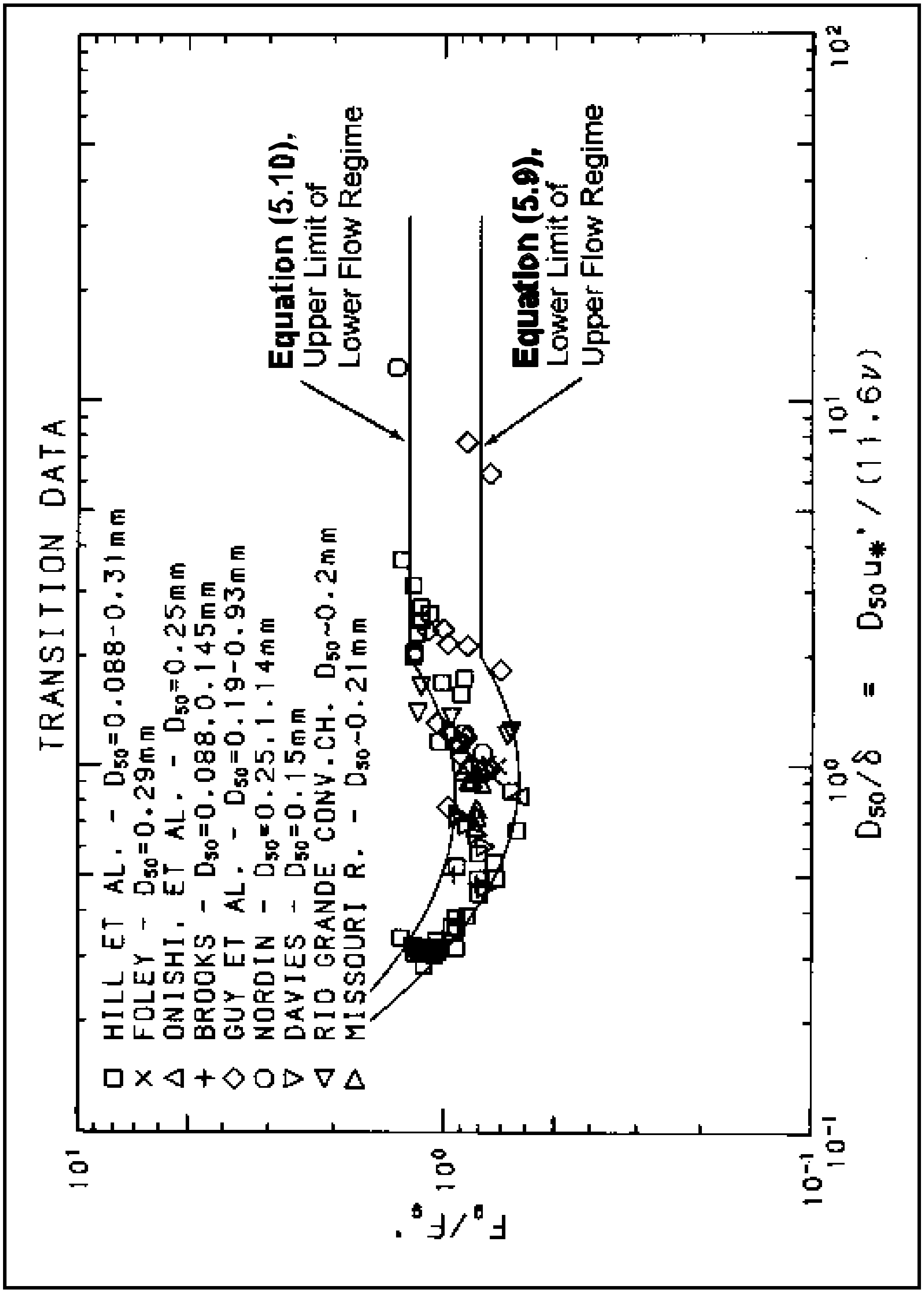


Figure 5.42 Viscous Effects on the Transition From Lower Flow Regime to Upper Flow Regime (from Brownlie, 1981)

$$\log \frac{F_g}{F_g'} = -0.2026 + 0.07026 \log\left(\frac{D_{50}}{\delta}\right) + 0.9330\left[\log\left(\frac{D_{50}}{\delta}\right)\right]^2 \qquad (5.10)$$

and log 0.8 for $D_{50}/\delta \geq 2$.

Brownlie (1981) summarizes his analysis for flow resistance determination by stating:

for slopes less than 0.006, only upper regime flow is expected. For slopes less than 0.006, the maximum velocity of the lower regime can be determined from $F_g = 0.8\ F_g'$ and the minimum velocity of the upper regime from $F_g = 1.25\ F_g'$.

Brownlie's analysis covered a wide range of conditions. His flow resistance equations applied to sand bed material ranging in size from 0.088 to 2.8 mm. The range of slope used in calibration was 0.000003 to 0.037.

Brownlie's equations relating hydraulic geometry and flow resistance are (5.11), (5.12), and (5.13).

Upper Regime:

$$\frac{R\ S}{D_{50}} = 0.2836\left(q_* S\right)^{0.6248}\ S^{\,0.0875}\ \sigma_g^{0.08013} \qquad (5.11)$$

Lower Regime:

$$\frac{R\ S}{D_{50}} = 0.3724\left(q_* S\right)^{0.6539}\ S^{\,0.09188}\ \sigma_g^{0.1050} \qquad (5.12)$$

where: R = hydraulic radius associated with the bed;
D_{50} = median grain size;
S = slope;
σ_g = geometric bed material gradation coefficient; computed by $(d_{84}/d_{16})^{1/2}$;
q = unit discharge; for wide channels assumed to be V*D; and
g = acceleration of gravity.

$$q_* = \frac{q}{\sqrt{g D_{50}^{\,3}}} \qquad (5.13)$$

The Brownlie sediment transport equation is used to relate hydraulic geometry to sediment concentration in the stable channel design method. The equation is taken to be equivalent to concentration measured as milligrams per liter with an error of less than 1 percent for concentrations

less than 16,000 ppm (Brownlie, 1981). The equation applies to sand ranging from 0.062 to 2.0 mm. Brownlie's (1983) sediment transport equation is as follows:

$$C = 7115\ C_F\ (F_g - F_{g_o})^{1.978}\ S_f^{\,0.6601} \left(\frac{R}{D_{50}} \right)^{-0.3301} \tag{5.14}$$

where:

$$F_{g_o} = 4.596\ \tau_{*c}^{\,0.5293}\ S_f^{\,-0.1405}\ \sigma_g^{\,-0.1606} \tag{5.15}$$

where: C = sediment concentration (ppm);
C_F = coefficient for field data; C_F = 1 for lab data and 1.268 for field data;
D_{50} = median grain size of the sediment particles;
S_f = slope of the energy grade line;
R = hydraulic radius;
τ_{*c} = critical Shields parameter; and
σ_g = $(d_{84}/d_{16})^{1/2}$

The critical Shield's parameter is calculated by Eq. (5.16), as defined by Brownlie:

$$\tau_{*c} = 0.22Y + 0.06(10)^{-7.7Y} \tag{5.16}$$

where:

$$Y = \left(\sqrt{\frac{\rho_s - \rho}{\rho}}\ R_g \right)^{-0.6} \tag{5.17}$$

where: ρ_s = density of the sediment particles;
ρ = density of water; and
R_g = grain Reynolds number defined by the following equation:

$$R_g = \frac{\sqrt{g D_{50}^{\,3}}}{\nu} \tag{5.18}$$

where: ν = kinematic viscosity of water;
D_{50} = median grain size; and
g = acceleration due to gravity.

The concentration calculated from Brownlie's sediment transport equation applies only vertically above the bed. The total sediment transport, computed in SAM, in weight per unit time is computed by the following equation:

$$Q_s = \gamma\ C\ B\ D\ V \tag{5.19}$$

where: Q_s = sediment transport in weight/time;
B = base width;
γ = specific weight of water;
C = concentration;
D = hydraulic depth; and
V = average velocity.

An average concentration for the total discharge is then calculated by:

$$\overline{\overline{C}} = \frac{Q_s}{0.0027\ Q} \tag{5.20}$$

where: C = concentration using the total discharge in ppm;
Q_s = sediment transport in tons/day; and
Q = discharge in ft^3/s.

5.3.6.5 Model Application

Copeland's stable channel design method simultaneously solves Eqs. (5.11), (5.12), and (5.14) to satisfy water and sediment continuity for a series of slopes and widths. The designer may then select any point along a curve plotted with width on the x-axis and slope on the y-axis. The minimum slope can be selected as an extremal hypothesis design according to Chang (1980):

> *For an alluvial channel, the necessary and sufficient condition of equilibrium occurs when the stream power per unit length of channel γQS is a minimum subject to given constraints, where γ = the specific weight of water; Q = discharge; and S = slope. Hence, an alluvial channel with water discharge Q and sediment load Q_s as independent variables tends to establish its width, depth and slope such that γQS is a minimum. Since Q is a given parameter, minimum γQS also means minimum channel slope.*

If the minimum slope design is desired it can be evaluated graphically using a stable channel curve. A stable channel curve is a plot of slope versus width, in which the minimum stream power design corresponds to minimum slope. An example of a stable channel curve is given in Figure 5.43.

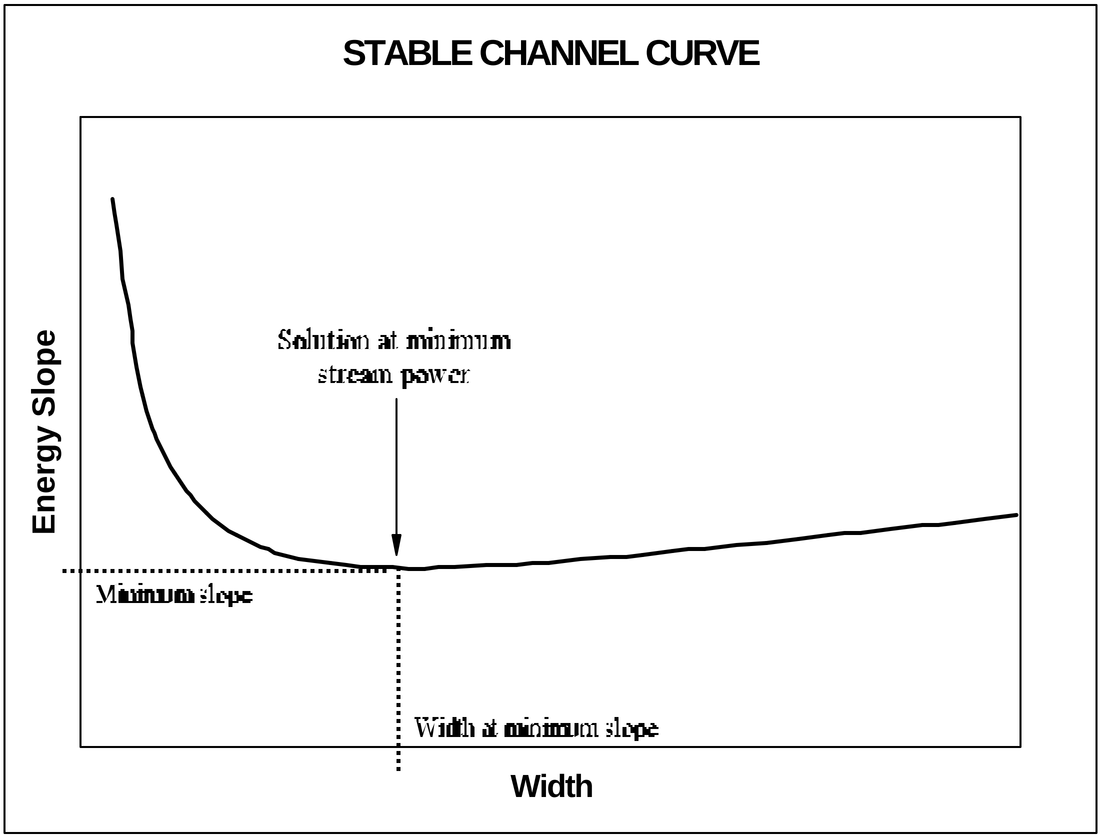


Figure 5.43 Example of a Stable Channel Curve

5.3.7 GRAVEL BED RIVERS

A method for stable channel design of alluvial sand bed rivers has been reviewed in Section 5.3.6.5. The method is included in the Hydraulic Design Package for Channels (SAM). A similar method that applies to gravel bed rivers is proposed by Firenzi (1998). Governing equations of flow resistance and sediment transport used by the method are reviewed. The formulation of the method into a computer model titled Gravel Bed Rivers (GBR) is then discussed.

5.3.7.1 Governing Equations for Stable Channel Design Procedure

Three relationships between channel geometry and flow resistance have been developed and used extensively. The resistance parameters have been termed the Darcy-Weisbach friction factor, f, the Chezy coefficient, C, and the Manning coefficient, n. The steady flow relationships associated with these resistance coefficients are given below:

$$V = \sqrt{\frac{8g}{f}} \cdot R^{1/2} S_f^{1/2} \tag{5.21}$$

$$V = C R^{1/2} S_f^{1/2} \tag{5.22}$$

$$V = \frac{\varphi}{n} R^{2/3} S_f^{1/2} \tag{5.23}$$

where: V = cross section averaged velocity;
R = hydraulic radius;
S_f = friction slope; and
φ = 1 for metric units and 1.486 for English.

Sediment laden flows are typically characterized by turbulent velocity profiles. This is true for gravel bed rivers in which there are more perturbations from rough boundaries. A turbulent velocity profile was developed by Prandtl (1926) using the defined relationship between shear stress and velocity gradient. In 1930, the velocity profile of Prandtl was verified by von Kármán. The velocity profile for turbulent flow near a plane boundary known as the Prandtl-von Kármán universal velocity distribution is written as:

$$\frac{v}{u_*} = \frac{1}{\kappa} \cdot \ln\left(\frac{z}{z_o}\right) \tag{5.24}$$

where: v = velocity at a point in the vertical;
u_* = shear velocity, defined as $(\tau/\rho)^{1/2}$;
κ = von Kármán constant ($\kappa \approx 0.4$);
z = vertical distance from channel bottom;
z_o = constant of integration;
τ = shear stress; and
ρ = density.

Equation (5.24) applies to a no slip boundary (i.e., turbulent velocity components vanish near the walls). This leads to viscous dominated flow in the location very near the boundary. The thin layer of laminar motion is known as the laminar sublayer. For the condition where the roughness elements are coarser than the laminar sublayer, the flow is termed hydraulically rough. Flow over gravel beds is considered hydraulically rough. For rough planes it has been determined that $z_o \approx \varepsilon/30$, where ε is the equivalent sand roughness from experiments by Nikuradse (1933). Integrating the Prandtl-von Kármán universal velocity distribution over the channel depth, h, and transforming the relationship into base-10 logarithm yields the Keulegan (1938) equation:

$$\frac{V}{u_*} = 5.75 \cdot \log\left(\frac{h}{\varepsilon}\right) + 6.25 \tag{5.25}$$

Equation (5.25) has been combined with Eqs. (5.22) through (5.24) to arrive at various logarithmic resistance relationships. Limerinos (1970) used the contributions of Leopold and Wolman (1957) and Chow (1959) to develop a relationship between Manning's n and relative smoothness:

$$\frac{n}{R^{1/6}} = \frac{0.0926}{a + b \log\left(\frac{R}{\varepsilon}\right)} \tag{5.26}$$

Limerinos found the smallest deviation between observed and computed values when D_{84} was used for the equivalent sand roughness (ε), where D_{84} is the size of the minimum particle diameter that equals or exceeds that of 84 percent of the river bed material. If D_{84} is used, the coefficients a and b obtain values of 0.76 and 2.00, respectively.

Often in gravel bed rivers, the banks do not have the same resistance elements as the bed. The bed resistance is due to a rough plane boundary and the bank resistance comes from vegetation or from soil that is different from that of the bed. Under these conditions it is ideal to calculate flow properties separately for the bed and banks.

Einstein (1942) proposed a method of separating the hydraulic radii of the bed and the banks. Lines perpendicular to the velocity contours are established that begin at the bed and end at the water surface. An example of such lines can be seen in Figure 5.44. There is no velocity gradient or shear stress across these lines. With the lines established, the cross section can be divided into three subsections. The total area of the cross section is related to the geometry of the subsections by Einstein (1950):

$$A_T = P_L R_L + P_B R_B + P_R R_R \tag{5.27}$$

where subscripts L, B, and R indicate left bank, bed, and right bank, respectively.

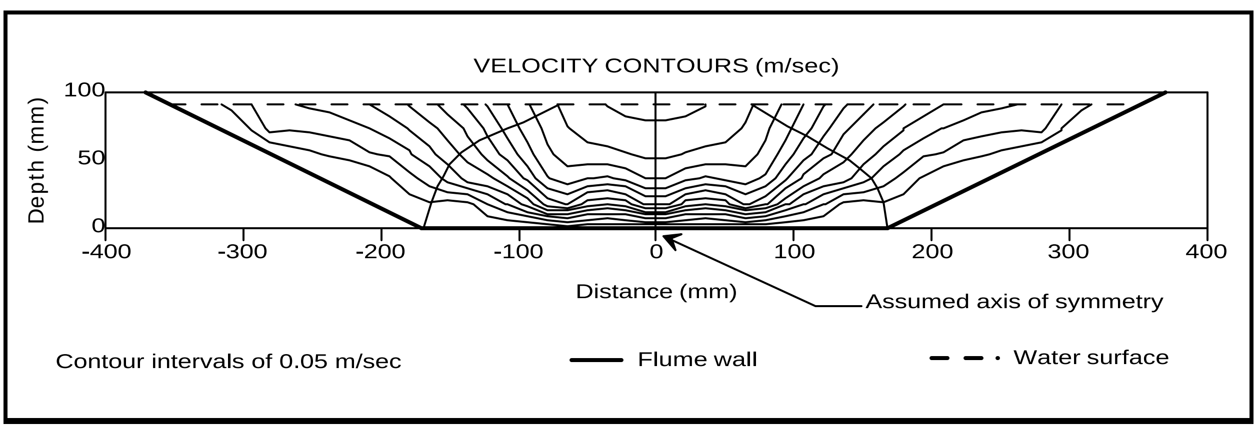


Figure 5.44 Velocity Contour Map With Lines Across Which There is No Shear Stress (after Gessler *et al.*, 1998)

Gravel river beds typically have a large distribution of particle sizes. Bed material will be transported at variable depths in the water column depending on the size of the particle and hydraulic characteristics of the flow. When gravel is being considered, there is typically two classifications of sediment discharge. Bedload refers to the portion moving on or near the bed of the river. Total load is defined as the total amount of sediment being transported (Biedenharn *et al*., 1997). The total load is comprised of the bedload and the portion being transported in suspension.

A dimensionless equation for the calculation of bedload discharge capacity was developed by Meyer-Peter and Müller (1948):

$$0.25 \cdot \left(\frac{\gamma}{g}\right)^{1/3} \frac{1}{(\gamma_s - \gamma)D_s} \cdot q_{bw}^{2/3} + 0.047 = \frac{Q_p}{Q}\left(\frac{n'}{n}\right)^{3/2} \frac{\gamma R S_f}{(\gamma_s - \gamma)D_s} \tag{5.28}$$

where: q_{bw} = unit channel width bedload transport in weight per time;
D_s = characteristic particle diameter;
γ_s = sediment dry unit weight;
γ = unit weight of water;
g = acceleration due to gravity;
n' = Manning's roughness associated with the grain resistance;
n = total Manning's roughness;
Q = water discharge; and
Q_p = portion of discharge contributing to bedload transport.

The Meyer-Peter and Müller bedload transport equation is based on extensive laboratory flume experiments. The range of sediment sizes used in calibration was 0.4 to 30 mm. The slope ranged from 0.0004 to 0.02.

Chien (1954) showed that the original elaborate Meyer-Peter and Müller bedload equation can be modified to give the following relationship:

$$\Phi = \left(\frac{4}{\psi} - 0.188\right)^{3/2} \tag{5.29}$$

where ϕ and ψ are parameters from Einstein (1942). These parameters are stated as follows:

$$\Phi = \frac{q_{bf}}{\gamma_s} \frac{1}{\sqrt{(G-1)gD_s^3}} \tag{5.30a}$$

$$\psi = (G-1)\frac{D_s}{RS_f} \tag{5.30b}$$

where: $(G-1)$ = $(\rho_s - \rho)/\rho$;

ρ_s = the density of the sediment; and

ρ = the density of water.

If Eq. (5.29) and Eqs. (5.30a and b) are combined a simplified Meyer-Peter and Müller bedload transport equation is given as:

$$\frac{q_{bv}}{\sqrt{(G-1)gD_s^3}} = 8(\tau_* - \tau_{*c})^{3/2} \tag{5.31}$$

where q_{bv} is the unit channel width bedload transport in volume per time. The term τ_* is known as the Shields parameter, which represents a dimensionless value of shear stress on the bed material. The Shields parameter is given by definition as:

$$\tau_* = \frac{\tau}{\gamma(G-1)D_s} \tag{5.32}$$

The term τ_{*c} is the critical Shields parameter. The critical Shields parameter is a threshold value of dimensionless shear stress where incipient motion exists. Often when working with gravel bed rivers $\tau_{*c} = 0.047$ is used. A more precise value can be determined using the modified Shields diagram, given in Figure 5.45, where τ_{*c} is the value selected from the regression line. However, the Shields diagram was developed using primarily uniform particle sizes, ignoring the effects of particle hiding and particle exposure.

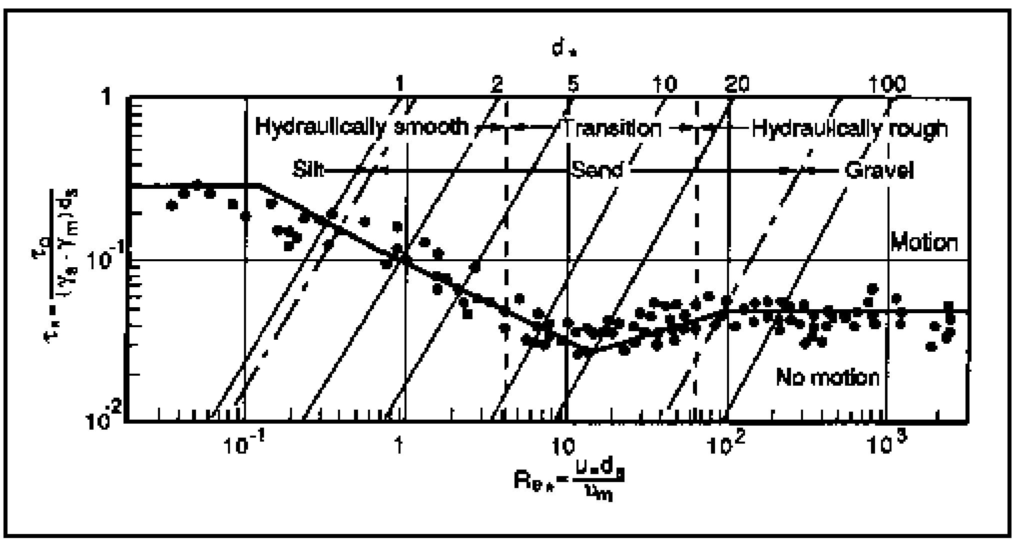


Figure 5.45 Threshold of Motion for Granular Material (from Julien, 1995)

For well-graded bed sediment or fine gravel, there may be enough material being transported in suspension that it is necessary to use a total load equation. Julien (1995) presents a plot showing the ratio of suspended load to total load versus u_*/ω, where ω is the particle fall velocity. It can be seen in Figure 5.46 that bedload is the dominant means of sediment transport for values of u_*/ω less than about 0.4. Under this circumstance, the Meyer-Peter and Müller equation is sufficient. If this criteria is not met, then a total load equation is necessary. Simons *et al.* (1981) developed a coarse grain total load transport

equation. The equation was intended for used in the arid environment of Pima County, Arizona. Due to the infrequent opportunities to measure sediment transport rates under flood conditions, the equation was theoretically derived. Meyer-Peter and Müller's equation was used for the bedload portion of the sediment transport. For the suspended portion, Einstein's method of integrating suspended load was used. Einstein's suspended load equation is written as:

$$q_s = q_b \cdot \left[I_1 \ln\left(\frac{30h}{D_s} \right) + I_2 \right] \tag{5.33}$$

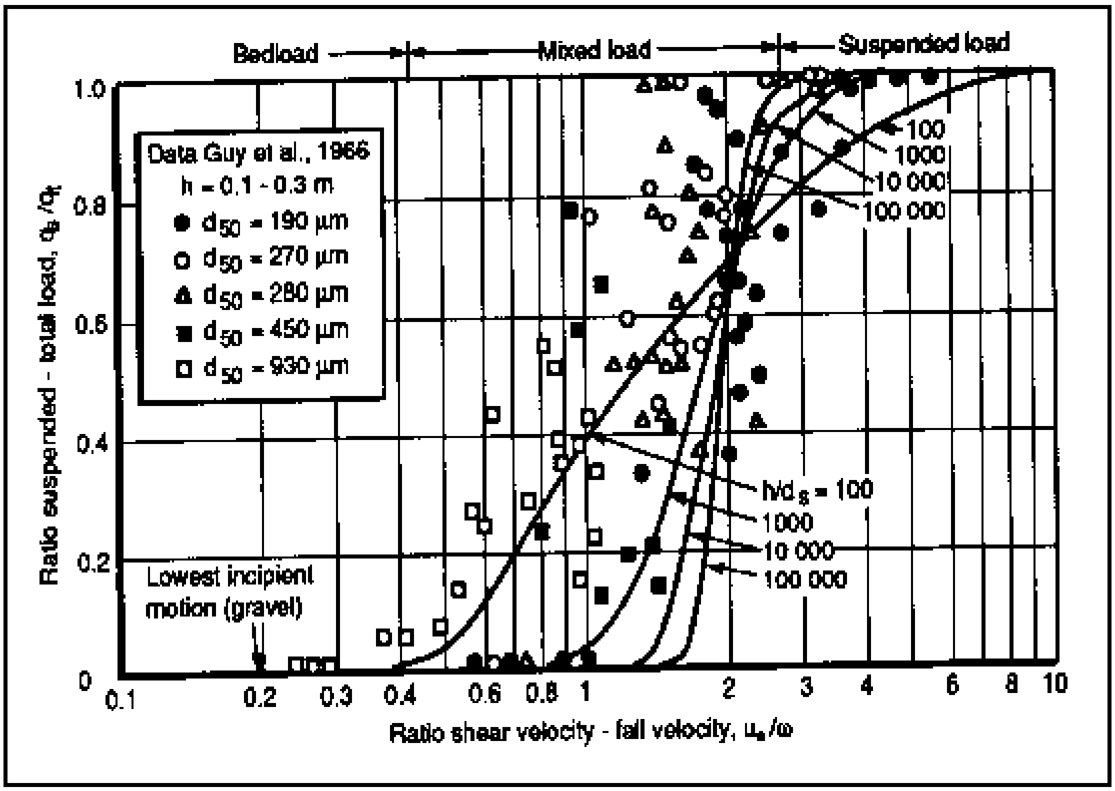


Figure 5.46 Ratio of Suspended to Total Load Versus Ratio of Shear to Fall Velocities (from Julien, 1995)

where q_b is the sediment transported in the bed layer with a thickness of $a = 2D_s$. The terms I_1 and I_2 are the Einstein integrals. Calculation of the integrals is a cumbersome task that can be performed numerically or with the use of nomographs prepared by Einstein (1950). The bedload and suspended load were calculated and combined under a variety of conditions. The range of particle size was 0.5 to 10 mm and the bed slope ranged from 0.001 to 0.04. The data were then used to calibrate a simplified equation for total load sediment transport. The equation, as presented in Zeller and Fullerton (1983), is:

$$q_{tv} = 0.0064 \cdot \frac{n^{1.77} V^{4.32} Gr^{0.45}}{d^{0.30} D_{50}^{0.61}} \tag{5.34}$$

where q_{tv} is the unit channel width total load transport in volume per time. The term Gr is the gradation coefficient calculated as follows:

$$Gr = \frac{1}{2} \cdot \left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}} \right) \tag{5.35}$$

Equation (5.34) is dimensional. The particle size, D_{50}, is in millimeters and all other variables are in standard English units.

5.3.7.2 Model Application

The stable channel design procedure developed by Firenzi (1998) simultaneously solves equations of flow resistance and sediment transport. The Manning equation is used as a relationship between roughness and hydraulic geometry. The program user may specify one of two sediment transport relationships: the Meyer-Peter and Müller bedload equation or the Simons, Li and Fullerton total load equation. These equations have been reviewed in the previous section. The program partitions the design cross section into three subsections according to Einstein (1950). Partitioning the cross section in this manner allows for Manning's n to be specified separately for the bed and banks, where the Limerinos (1970) equation is used for the channel bed. Allowing Manning's n to be different for the bed and banks of the channel makes the program applicable to small rivers where it is not valid to neglect the effects of bank roughness.

Three degrees of freedom are assumed in the method: width, depth, and slope. In the absence of a third equation to satisfy the three degrees of freedom, a table of solutions is generated by the program. It is left to the user to incorporate a third governing principle. The minimum slope can be selected as an extremal hypothesis design according to Chang (1980). Note that minimum slope corresponds to minimum stream power when a single design discharge is used. A stable channel curve can be used to graphically evaluate the design at minimum slope. Figure 5.43, presented in Section 5.3.6.5 is an example of a stable channel curve.

5.3.8 HEC-6

HEC-6 is a one-dimensional numerical model designed to simulate and predict changes in river profiles due to scour and/or deposition over average time periods. The model is based on movable boundary, open channel flow hydraulics with time periods normally in years, although single flood events with days or months are also possible. Various features in HEC-6 include: network stream analysis capability, channel dredging, assortment of levee and encroachment alternatives, and several methods for computing sediment transport rates (USACE, 1993). The following sections present an overview of the computational process and the four input categories: geometry and hydraulics, sediment, hydrology, and special commands.

The computation process begins by partitioning a continuous flow record into a series of steady flows with variable discharges and durations, i.e., composing a discharge hydrograph. Starting with the first flow in the hydrograph, a water surface profile is calculated. The water surface profile provides information for each cross section, such as the energy slope, velocity, depth, etc.

Potential sediment transport rates are then calculated at each cross section. Combining the sediment transport rates with the duration of flow gives a volumetric summary of sediment within each reach. Sediment calculations use grain size fractions which allow the simulation of hydraulic sorting and armoring. The amount of scour or deposition at each cross section is then computed and the cross section geometry is adjusted accordingly. The computations move to the next flow in the hydrograph and the cycle is repeated using the updated geometry (USACE, 1993).

Geometry data are represented by cross sections comprised of station-elevation coordinates, distances between cross sections, and Manning's n-values. The movable bed portion of each cross section and the depth of sediment material in the bed are also defined. HEC-6 raises or lowers cross section elevations to show deposition or scour. Horizontal locations of the channel banks are considered fixed. Floodplains on both sides of the channel are considered to have fixed ground locations but can be moved vertically if within the movable bed limits specified by the user. Left and right overbank stations are defined in the geometry data, as well as any ineffective flow areas or containment of flow by levees (USACE, 1993).

The one-dimensional energy equation is solved by the standard step method and used to compute the water surface profiles for each flow in the hydrograph. Downstream water surface elevations must be determined for each discharge in the hydrograph by either a rating curve specified by the user, or a time dependent water surface elevation.

Sediment data includes the fluid and sediment properties, inflowing sediment load, and the gradation of the stream bed material. Sediment transport rates may be calculated for grain sizes up to 2,048 mm. Particle sizes larger than 2,048 mm existing in the bed material are used for sorting computations but are not transported. Sediment transport functions used to calculate the bed material load are specified by the user. Numerous sediment transport functions available in HEC-6 are available (USACE, 1993).

Thomas (1996) developed a HEC-6 simulation of Hotopha Creek, one of the DEC streams. The results of that investigation indicated that a reduction in sediment yield of 16% resulted from the construction of a series of grade control structures along Hotopha Creek.

During a 30-year simulation of the Hotopha Creek watershed, the results suggested that channel degradation may resume downstream of several drop structures because of the success of those structures in halting upstream erosion. The advantage of long-term simulation to check grade control and other erosion prevention features is readily evident. When the goal of a project is to reduce sediment yield, and the project is successful, the channel reaches downstream of the project will be susceptible to degradation. HEC-6 modeling of the complete channel system in a watershed allows channel spacial and temporal response to be predicted.

5.3.9 BANK STABILITY

Natural and excavated stream banks often need to be analyzed for stability. Historically, soil mechanic approaches to stability have been applied to stream banks. The instability and subsequent failure of stream banks commonly result from a combination of hydraulic, geomorphic, and geotechnical factors. A meandering channel produces both vertical and horizontal hydraulically driven scour on the outside of channel bends. As scour occurs the bank height increases, which typically results in the failure of the bank. Although the analysis of bank stability may be completely geotechnical, design of any hydraulic structure to reduce bank failures along a channel requires consideration of hydraulic, geomorphic, and geotechnical factors.

The instability of stream banks results in a geotechnical failure of the slope. A geotechnical failure involves the movement of relatively large and possibly intact segment of soil. There are many different ways that stream banks may fail. However, there are two distinct classes of bank failure: the slow moving creep failure, and the catastrophic shear failure. Within the DEC watersheds only the catastrophic shear failure is considered, since creep failures may take years to be recognized. Shear failure is based on the mechanics of the failure. Rotational and slab-type block are the most commonly observed within the DEC watersheds. Streambank and erosion processes are discussed in more detail in Section 3.4.3.2.

Rotational failures are usually associated with a circular, or log spiral failure plane. Rotational failure is associated with high gentle slopes, and bank angles less then 60 degrees to the horizontal. Bank angle less then 60 degrees to the horizontal are considered mild slopes.

Planar slip failures are commonly associated with lower, steep banks and the failure plane is more linear then the rotational failure plane. Bank angles associated with the planar sliding failures are usually greater than 60 degrees to the horizontal and the slope is considered steep.

Whether analyzing the stability of mild or steep slopes, the approach taken often depends upon the objective of the investigation. For example, the analysis of a low-head earthen dam may warrant a detailed study using a finite difference approach, yet a large riverbank having roughly the same general shape and size may be analyzed using stability curves. Ultimately, the approach and the level of detail of bank stability analysis is governed by the available information and the time allotted to determine stability. Several essential requirements for conducting detailed stream bank stability analysis include the following:

- choosing the correct method of analysis;
- accurate description of the bank geometry;
- reliable soil properties;
- correct description of slope hydrology, i.e. groundwater table and seepage conditions; and
- correct definition of external loads, i.e. surcharges, line loads, earthquake loads.

These requirements are sometimes difficult to observe or obtain, and the lack of information or an incorrect selection of method may yield poor results.

Numerous methods are available for predicting the stability of stream banks. Most methods used employ the concept of limited equilibrium analysis. The use of limited equilibrium analysis explicitly accounts for the major factors that influence the shear stress and shear resistance of a slope, and employs the comparison between resisting forces (F_R) and driving forces (F_D). The slope under consideration is considered stable as long as the resisting forces are greater then the driving forces ($F_R > F_D$). Both mild and steep slopes can be analyzed employing the concept of limited equilibrium analysis.

The resisting force, F_R, is derived from the shear strength of the soil, and keeps the slope from moving. The shear strength of a soil is defined by Eq. (5.36):

$$\tau = c + \sigma \tan \phi \qquad (5.36)$$

where: τ = shear strength of the material;
c = cohesion intercept;
σ = normal stress on the failure surface; and
ϕ = angle of internal friction.

Equation (5.36) is known as the Revised Coulomb Equation. The angle of internal friction (ϕ) and cohesion (c) are known as the shear strength parameters. Each shear strength parameter can be determined from laboratory tests on soil samples or back-calculated after failure of a stream bank occurs.

Forces tending to cause movement of the slope, or the driving forces (F_D) include the weight of the soil mass and any external loading. External loading may be additional loading on the top bank or a surcharge of pore-water pressure. The ratio between resisting and driving forces define the factor of safety (FS) and is determined by Eq. (5.37):

$$FS = \frac{F_R}{F_D} \qquad (5.37)$$

The factor of safety can also be considered as the ratio of the critical bank height to the actual bank height, as represented by Eq. (5.38).

$$FS = \frac{H_c}{H} \qquad (5.38)$$

where: H_c = critical bank height; and
H = actual bank height.

Failure is anticipated when the factor of safety is less then unity. Figures 5.47a and 5.47b depict the fundamental failure geometry associated with limited equilibrium bank instability for a low steep

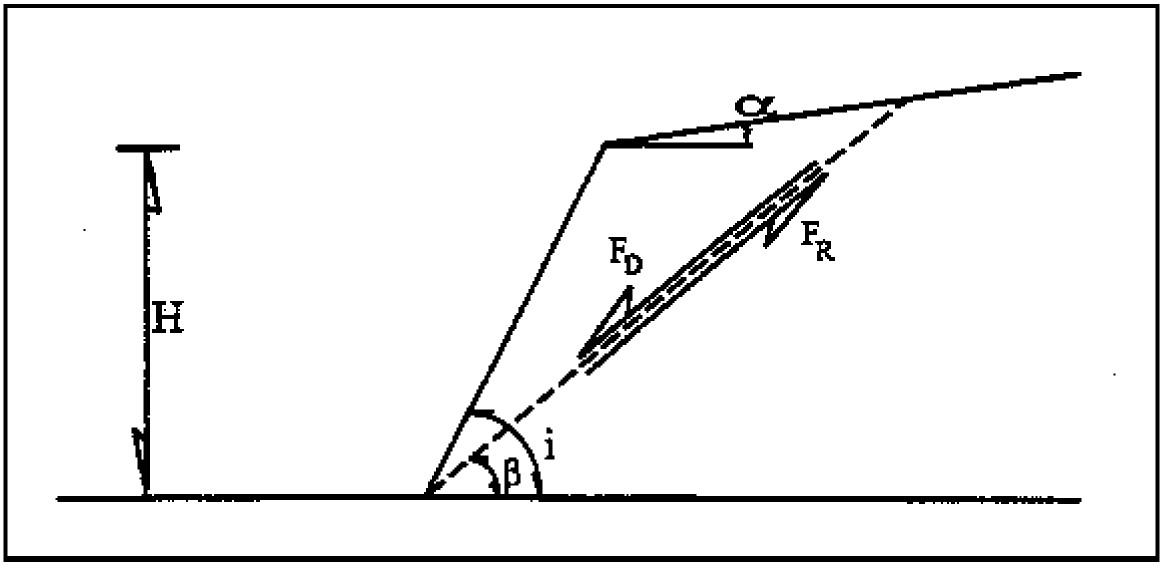


Figure 5.47a Shear Failure Along a Planar Slip Surface Through the Toe of the Slope

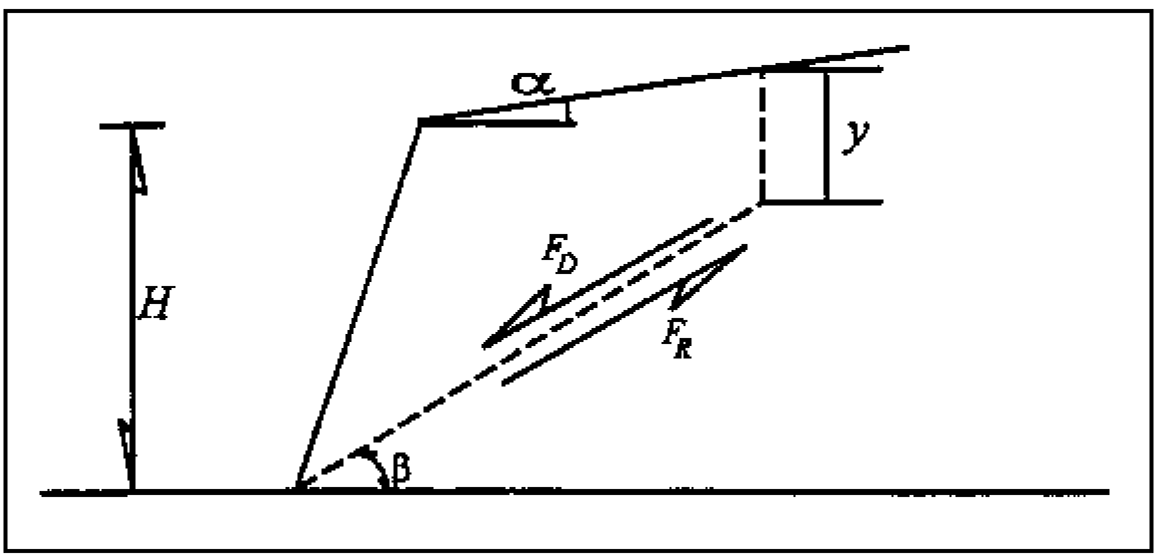


Figure 5.47b Shear Failure Along a Planar Slip Surface Through the Toe of the Slope With a Tension Crack

slope, prone to fail in a planar fashion. Equations (5.39) and (5.40) define the driving force (F_D) and resisting force (F_R) components of the factor of safety, respectively.

$$F_D = W \sin \beta \tag{5.39}$$

$$F_R = c' L + N \tan \phi' \tag{5.40}$$

where: W = weight of a unit width of bank;
â = failure plane angle in degrees;
$c^{)}$ = effective cohesion;
L = length of the failure plane;
N = normal force; and
$M^{)}$ = friction angle of the soil.

Often, just prior to failure, a tension crack will develop parallel to the stream bank and can be observed from the top bank. A tension crack is a vertical separation of the soil resulting in a cavity or crevice. Vertical tension cracks at the surface of a slope, possibly occurring along natural cleavage planes, reduce the overall stability of a slope. The presence of tension cracks reduces the critical bank height. At failure, tension cracks may quickly develop to depths greater than half the bank height. As a conservative measure, Thorne and Abt (1989) recommend using a tension crack depth of half the bank height if no site-specific data are available. Generally, varying a tension crack depth from 30 to 50 percent of the bank height is a realistic range and does not change the factor of safety by more then 10%.

5.3.9.1 Required Geotechnical Data

Bank stability determination relies heavily on soil properties. Review of the principal equation governing limited equilibrium analysis indicate that the shear strength of a soil, and subsequently the resisting force, is based on the cohesion and the angle of internal friction of a soil. The driving force is based on the weight of the soil, and is a function of the failure block geometry and the unit weight of the soil. The three soil properties required for bank stability calculations in the DEC watersheds are cohesion, unit weight, and internal friction angle. These three soil properties, bank height, and bank angle are the minimum requirements for slope stability calculations. The methods currently employed to determine the stability of stream banks within the DEC watersheds require a composite or average values of cohesion, unit weight, and internal friction angle.

An important consideration in stream bank stability analysis is whether to employ a total or and effective stress analysis. A total stress analysis using undrained shear strength parameters (c,ö) is limited to slopes where pore pressures are governed by external stress changes. These conditions are characteristic of post-construction problems. A total stress analysis does not require a determination of pore pressure in the bank and is an important advantage for a total stress analysis. An effective stress analysis is warranted when pore pressures are governed by steady state seepage conditions, or if long-term stability is a consideration. Steady state seepage is the usual condition for natural stream banks. Effective stress parameters $(c^{)},N^{)})$can be determined from either drained or undrained triaxial tests with pore pressure measurements. However, if the pore pressures within a stream bank are unknown or cannot be determined, there is little point to an effective stress analysis, and a total stress analysis should be employed.

5.3.9.2 Soil Data Sources

During the design of many hydraulic structures by the Vicksburg District U.S. Army Corps of Engineers, it is standard practice to perform a geotechnical analysis. Soil characteristics are obtained by extracting soil borings and analyzing the soil in the boring. In most instances, soil borings are stratified consisting of several layers. Each layer, or strata, of soil has unique physical properties. Laboratory tests are conducted on each layer of soil to determine a variety of physical properties including; moisture content, percent organic, gradation, internal friction angle, cohesion, and dry unit weight. In addition to the soil boring, a geotechnical analysis usually includes local physical properties like the average bank height.

Within the Yazoo River Drainage basin in northern Mississippi there are six sub-basins, to include: Batupan Bogue, Black Creek, Coldwater River, Hickahala Creek, Hotopha Creek, and Long Creek. Design plans and as-built drawings for hydraulic structures, scattered throughout these six sub-basins, were reviewed in 1997 to obtain soil-boring information. Design plans, as-built drawings and soil boring results were primarily obtained from the Agricultural Research Service in Oxford, Mississippi. The U.S. Army Corps of Engineers Waterways Experiment Station in Vicksburg, Mississippi provided the remaining soil data.

5.3.9.3 Soil Data Evaluation

Currently, slope stability estimations are based upon single values of internal friction angle, cohesion, and unit weight. Sixty-five soil borings were obtained within the Yazoo River Drainage basin. The bank averaged soil properties, for each soil boring, were obtained by:

- determining the bank height to be used in the averaging process;
- determining the percentage of bank in each strata;
- multiplying the percent of bank by the internal friction angle, cohesion, and unit weight; and
- sum the percentages to obtain the bank-averaged values.

Table 5.10 is an example of the soil averaging process. Soil-boring data was limited for the Long Creek sub-basin. However, Thorne (1988) conducted field investigations in the Long Creek sub-basin to obtain soil properties. Thorne (1988) reported using the same averaging procedure for the determination of soil properties. Averaged soil properties from each boring where collected and then combined on a sub-basin level basis to obtain sub-basin average properties.

Significant variability in soil properties even within sub-basins was observed. Despite this variability, with no site specific soil data the sub-basin averaged soil properties are the most logical values to be used in slope stability. Table 5.11 summarizes the maximum, average, and minimum values of internal friction angle, cohesion, and unit weight for the six sub-basins.

Table 5.10 Soil Properties and the Averaging Method

The natural bank height is 18.3 feet

Soil Description	Depth (ft)	Friction Angle M) (degrees)	Cohesion c) (lbs/sf)	Unit Weight (lbs/cf)
Silty clay	0 - 4.5	14	300	115
Clayey sand	4.5 - 7.5	17	700	140
Clay	7.5 - 15.0	15	550	130
Stiff clay	15.0 - 26.0	17	1,100	145
Clay	---	---	---	---

	% of Bank Height	Friction Angle M) (degrees)	Cohesion c) (lbs/sf)	Unit Weight (lbs/cf)
	19.3	2.70	57.94	22.21
	12.9	2.18	90.13	18.03
	32.2	4.82	177.04	41.85
	35.6	6.05	391.85	51.65
Total	***100***	***15.78***	***716.95***	***133.73***

5.3.9.4 Stability of Mild Slopes

The majority of the streams within the Yazoo River Drainage basin have mild stream bank slopes. Mild slopes are less then 60 degrees to the horizontal. To determine the stability of these mild slopes the DECBank computer program was developed.

The DECBank computer program reads and interprets HEC-2 input data files to determine the required bank heights and angles for stability calculations. Users of DECBank have the ability to visually inspect each cross-section prior to stability calculations to ensure that they agree with the determined bank angles. If the user does not agree with the computer determined bank angle, they have the ability to alter the bank angles and visually inspect the new bank angle of the natural cross section.

Mild slope stability (bank angles between 30 and 60 degrees) is determined by use of numerical representations of Osman's (1985) stability curves. Osman's (1985) stability curves were developed using numerous stability computations based upon the simplified Bishop method of slices. Singular or average values of internal friction angle, cohesion, and unit weight are used in conjunction with the equations representing Osman's (1985) stability curves to determine the stability of both banks of a cross section. Osman's (1985) stability curves are dimensionless, so the DECBank computer program is applicable for both English and Metric units. After determining the stability of all appropriate cross sections DECBank automatically determines the average factor of safety for the entire river reach.

Table 5.11 Soil Properties

Sub-Basin Name	Maximum Cohesion (lbs/sf)	Maximum Unit Weight (lbs/cf)	Maximum Friction Angle (degrees)	Minimum Cohesion (lbs/sf)	Minimum Unit Weight (lbs/cf)	Minimum Friction Angle (degrees)	Summary Cohesion (lbs/sf)	Summary Unit Weight (lbs/cf)	Summary Friction Angle (degrees)
Batupan Bogue	2,547.9	125.2	27.9	62.5	87.9	10	894	110	17
Black Creek	2,007.8	126.2	13.5	387.0	91.4	3.7	890	108	10
Coldwater River	3,133.9	110.3	20.0	210	80.9	2.1	1,435	99	9
Hickahala Creek	3,020.0	125.0	20.0	204.5	121.6	0.0	788	120	8
Hotopha Creek	3,593.2	123.9	20.0	145.5	100.0	0.0	998	117	13
Long Creek	---	---	---	---	---	---	331	121	14.7
Yazoo River							889	113	12

--- not available

5.3.9.5 Stability of Steep Slopes

Currently, the Thorne (1988) model is used to determine the stability of steep stream bank slopes. Slopes greater then 60 degrees to the horizontal are classified as steep.

The Darby and Thorne (1996a,b) model utilizes a similar interface as the DECBank program, in that it reads and interprets HEC-2 input data files to develop the required bank heights and angles for stability calculations. Unlike the DECBank, the Darby and Thorne (1996a,b) model can directly account for pore and confining water pressures, in the determination of the factor of safety of a stream bank. The Darby and Thorne (1996a,b) model performs all stability calculations in Metric units, so the user is required to convert or obtain singular or average values of cohesion and unit weight to or in Metric units.

CHAPTER 6

SELECTION AND DESIGN OF CHANNEL REHABILITATION METHODS

This chapter addresses the selection and design of channel rehabilitation methods. The types of improvement measures adopted in a rehabilitation project depend upon the goals of the project and the physical characteristics of the watershed. The three principle techniques discussed in this chapter are grade control, bank stabilization, and flow control. Bank stabilization and grade control are the primary methods employed in channel rehabilitation projects to control erosion and sedimentation. Because channel rehabilitation projects often occur in urbanized areas where flow control has been implemented, it is important to integrate the morphologic impacts of these features into the channel rehabilitation plan. For this reason, a discussion of the morphologic response to flow control is presented in this chapter.

6.1 STREAMBANK STABILIZATION

Although there are many different types of bank stabilization measures, they can generally be classified as armor protection, indirect protection, or vegetation. General descriptions, advantages, disadvantages, and typical applications are presented in this chapter. For a more detailed treatment of streambank stabilization, the reader is referred to Biedenharn *et al*. (1997).

The suitability and effectiveness of a given bank stabilization technique are a function of the inherent properties of that technique, and in the physical characteristics of the proposed worksite. Consequently, there is no single stabilization technique that is applicable to all situations. Unfortunately, many practitioners often attempt to force a particular technique that they are familiar with into all situations. For instance, there are some engineers and scientist that believe that bioengineering techniques are the answer to all erosion problems. Likewise, there are those who will recommend a complete riprap armor for the bank when another, less costly, and perhaps more environmentally acceptable technique would be just as effective.

Although there is very little guidance available for establishing the suitability of a particular technique for a particular site condition, the designer should make the selection within the framework of three criteria:

Effectiveness of the alternative;
Environmental considerations; and
Economic factors.

Many techniques can be designed to adequately solve a specific bank stability problem by resisting erosive forces and geotechnical failure. The challenge to an engineer is to determine the most effective solutions to a specific problem, by recognizing which technique matches strength of protection against strength of attack, and which therefore performs most efficiently when tested by the strongest process of erosion and most critical mechanism of failure. Environmental and economic factors are integrated into the selection procedure, but the chosen solution must first fulfill the requirement of being effective as bank stabilization, otherwise environmental and economic attributes will be irrelevant.

6.1.1 SURFACE ARMOR

Armor is a protective material in direct contact with the streambank. It must have sufficient weight and/or strength to remain in place when subjected to hydraulic forces and impact from objects carried by the stream. It must also prevent significant loss of bank material from under the armor due to turbulence of flow or movement of groundwater.

Armor is often simply called revetment, but the more specific term armor is used here because other forms of bank stabilization, such as retards and retaining walls, are also referred to in some regions as revetments.

Armor materials can be categorized as follows:

Stone;
Other self-adjusting armor;
Rigid armor; and
Flexible mattress.

Armor protection requires careful consideration of the geotechnical stability of the bank, and sometimes a granular or fabric underlayment is required for proper interior drainage of the bank material, or to prevent loss of fine grained material through the armor.

6.1.1.1 Stone Armor

Stone armor is the most commonly used type of armor protection. There are many variations in the design of stone armors. The riprap blanket is the most recognizable form of stone armor. It is often

considered the benchmark against which other bank stabilization techniques are judged, not only because it can be designed to solve almost any problem, but because it can be designed precisely, thus its performance and cost can be predicted more reliably than for other methods. Other commonly used forms of stone armor include: trenchfill revetment which is simply a standard stone armor blanket with a massive stone toe constructed in an excavated trench behind the river bank, in anticipation that the river will complete the work by eroding to the revetment, causing the stone toe to launch down and armor the subaqueous bank slope; windrow revetment which is simply an extreme variation of a trenchfill revetment consisting of rock placed on the floodplain surface landward from the existing bankline at a pre-determined location, beyond which additional erosion is to be prevented; and longitudinal stone toe which is another variation of the windrow revetment with the stone placed along the existing streambed rather than on top bank.

Some armor materials other than stone which have the ability to adjust to scour, settlement, or surface irregularities are: concrete blocks; sacks filled with earth, sand, and/or cement; and soil-cement blocks. Armor materials which have been occasionally used in the past, but which have serious engineering and environmental shortcomings are: rubble from demolition of pavement or other source; slag from steel furnaces; and automobile bodies.

Advantages: Because its performance has been so thoroughly analyzed by research and practical application in a wide range of conditions, stone armor can be designed with an especially high degree of precision and confidence. The American Society of Civil Engineers' Task Committee on Channel Stabilization Works stated in 1965 that:

> *Stone is the most commonly used material for upper bank paving for revetment works, and in most cases has proved superior to other materials because of durability and ability to conform to minor irregularities in the slope (ASCE, 1965).*

Since that time, further development and application of manufactured proprietary armor materials, and increasing emphasis on environmental considerations and the use of vegetation for erosion control, has tempered that observation to some degree. However, the favorable attributes of stone armor are not diminished by the increasing availability of alternative materials. Furthermore, well-graded stone can often be placed without a separate underlayment material, because it provides permeability without exposing bank material. This characteristic may be a crucial factor when comparing the economics of alternative armor materials.

Disadvantages: Stone may be more costly than other materials, depending on its availability. It requires heavy equipment for efficient placement on large projects. It may be considered unaesthetic for some locations, and may not compare favorably with other materials in some environmental circumstances.

6.1.1.2 Rigid Armor

Rigid armor is an erosion-resistant material which has little or no flexibility to conform to bank irregularities occurring after construction. Typically, the armor is placed directly on the bank slope in a fluid or chemically reactive state, then hardens.

The most common rigid armors are: asphalt; concrete; grouted riprap (or other grouted armor material); and soil-cement. Materials which have a more restricted use, but which can be classified as rigid armors, are chemical soil stabilizers, and clay.

Rigid armor in the form of concrete, asphalt, or grouted riprap is often considered for use in situations where high velocities or extreme turbulence make adjustable armor ineffective or very expensive. Typical uses are in conjunction with hydraulic structures or in artificial channels on steep slopes.

Rigid armor may be the preferred alternative in flood control or drainage channels where low boundary roughness is mandatory, or in water supply channels where prevention of water loss due to infiltration into the bank is important. It is suitable for bank slopes which must be easily traversed by pedestrians or recreational users, if the slope is not too steep for safety.

Rigid armor is sometimes the least costly alternative, typically where adjustable armor is not available locally, especially if a geotechnical analysis of the bank material indicates that elaborate subsurface drainage work is not necessary.

Advantages: The most common rigid armors will withstand high velocities, have low hydraulic roughness, and prevent infiltration of water into the channel bank. They are practically immune to vandalism, damage from debris, corrosion, and many other destructive agents. The most common rigid armors are easily traversed by pedestrians.

Disadvantages: A rigid armor requires careful design and quality control during construction, and unfavorable weather conditions can cause construction delays. Chemical soil stabilization, and clay have a limited range of effectiveness.

Provision for draining groundwater and preventing the buildup of excess positive pore water pressures, in the form of a filter or subsurface drains, must usually be provided for impermeable armors, which may significantly increase the cost of the project.

Most rigid armors are difficult or impossible to construct underwater, although this difficulty can be alleviated for concrete by using one of the commercially available fabric mattresses.

Rigid armor, being inflexible, is susceptible to breaching if the bank material subsides or heaves. Increased wave runup on a smooth rigid armor may be a concern for some projects.

Some of these materials have little to offer environmentally, being biologically sterile and perhaps unacceptable aesthetically, depending on the surroundings.

6.1.1.3 Flexible Mattress

The basic concept of a flexible mattress is that material or objects which cannot resist erosive forces separately can be fastened together or placed in a flexible container to provide adequate resistance to erosive forces, while partially retaining the desirable characteristics of adjustable armor, especially that of flexibility.

The most common flexible mattress materials are: concrete blocks; fabric; and gabions. Materials which have a more limited use are: grids (for confining earth or other fill material); used tires; and wood.

This compromise between adjustable armor and rigid armor is most attractive when economical materials can be used for the mattress. In fact, the origin of some variations can be traced directly to creative use of local materials. Where no protective material of local origin is adequate to withstand the erosive forces in a given application, the most suitable method may be the one which requires the least amount of costly imported material, a requirement which is often met by a flexible mattress.

Advantages: Flexibility to adjust to scour or settlement and still remain in contact with the bed and bank is the most obvious trait. Most mattress materials which are sold under trade names share another advantage - they are available in various configurations, thus can be applied to a variety of situations.

Flexible mattresses can be placed underwater with a relatively high degree of confidence. If properly anchored to a geotechnically stable bank, they can be placed on steep slopes. They can be walked upon easily, thus are suitable for slopes used by pedestrians.

Disadvantages: Mattress components are subject to deterioration from the elements and vandalism. However, the damage is often within acceptable limits and since the various types are affected differently, identification of the hazards enables the designer to select an appropriate mattress for a given application. The construction of some types of mattresses are labor intensive, and may require skills not commonly available. However, the labor intensive aspect may not be a disadvantage in all cases, and may be an advantage in some cases.

6.1.2 INDIRECT TECHNIQUES FOR EROSION PROTECTION

Indirect protection structures extend into the stream channel, and redirect the flow so that hydraulic forces at the channel boundary are reduced to a non-erosive level. Indirect protection techniques can be classified as follows: dikes and retards; and other flow deflectors such as bendway weirs and Iowa vanes.

6.1.2.1 Dikes and Retards

Dikes are defined as a system of individual structures which protrude into the channel, generally transverse to the flow. Other terms which are often used are groins, jetties, spurs, wing dams, and if they protrude only a short distance into the channel, hard points. The term dikes is also used in some regions to refer to earthen flood-containing structures, which are also called levees, but that usage is not relevant here.

Retard is defined as a continuous structure approximately parallel to the streamflow. It can be a single structure or two, or more, adjacent and parallel structures, in which case the space between may be filled with various materials. Other terms that are sometimes used are longitudinal dikes, parallel dikes, jetties, guide banks, and training walls. Most designs have occasional tiebacks extending from the bank out to the main structure. These tiebacks have the appearance of dikes. In fact, many retard designs can be viewed as being a dike system with a longitudinal component connecting the ends of the dikes.

Dikes and retards can be applied to a wide range of conditions. However, the most common use is on shallow, wide streams with moderate to high transport of suspended bed material. Shallow channel depths reduce the required height of structures, a wide channel provides room for the channel alignment and geometry to adjust, and a heavy supply of suspended bed material accelerates the rate of induced deposition.

Where long-term funding is provided, dikes and retards are often built in increments in order to reduce costs by modifying the river form gradually, and taking advantage of subsequent deposition.

Dikes and retards can be used where establishment of riparian vegetation is a high priority. Initial plantings and natural establishment of native species can be supplemented by later plantings on sediments deposited within and behind the structures, or by sloping and vegetating the upper bank slopes once lower bank stability has been attained.

No formal and widely tested design criteria for dikes and retards exist, although design concepts based on experience and model tests have been developed for some applications. A study performed for the U.S. Federal Highway Administration and reported by Brown (1985) is one of the most comprehensive analyses of dikes. That report is based on model tests, a literature review, and a survey of several hundred field installations. Studies by the U.S. Army Corps of Engineers (USACE, 1981) also provide observations on design parameters.

Advantages: Dikes and retards provide a means to modify the channel alignment, are well suited to the incremental construction approach, and are amenable to the establishment of woody vegetation. Also, many designs use locally available material.

Dikes and retards offer the opportunity for incorporating a wide variety of environmental features by increasing the diversity of aquatic and terrestrial habitat, although subsequent sediment deposition may be detrimental to shallow water habitat. The reduction of water surface area due to deposition within the dike or retard system will reduce evaporation rates, which may be considered to be a benefit in semi-arid areas.

Dikes are usually less expensive than retards for a given situation, and will not interfere with access to the stream. Also, after the stream has adapted to the initial project, dikes can be extended farther into the stream if necessary to fully achieve project objectives, whereas with retards, modification of the initial alignment is likely to be much more expensive.

Disadvantages: Those designs that involve perishable materials or mechanical connections are susceptible to gradual deterioration and to damage by debris, fire, ice, and vandals.

Channel capacity at high flow is decreased initially when dikes or retards are constructed, although the channel will usually adjust by forming a deeper, narrower cross-section, and the ultimate result may even be an increase in conveyance capacity. However, the extent of the adjustment cannot be always be predicted reliably, even with physical or numerical models. Since conservative assumptions on future deposition and vegetative growth would be necessary, extensive use of dikes or retards must be approached with caution on projects where channel flood conveyance is a concern.

Dikes are more vulnerable to floating debris than are retards, since dikes present abrupt obstacles to flow, whereas retards, being approximately parallel to flow, will allow much of the floating debris to pass through the project reach. Also, erosion between the dikes in a system will often be more severe and of longer duration than erosion within a retard system.

6.1.2.2 Other Flow Deflectors

Structures other than dikes and retards may provide a means of altering hydraulic conditions in order to resist bank erosion in bends. One of the most intractable problems of river engineering is posed by the coupled processes of deposition of sediment on point bar faces and scour in the thalweg of bends. Several approaches have successfully addressed these coupled processes in some cases. These approaches alter secondary currents so that sediment transport away from the toe of the bank is reduced. This results in a more uniform cross-section shape, with shallower thalweg depths and a wider channel at low flow. These approaches include Iowa vanes, and bendway weirs. Because these are recently developed techniques, the long term success of these structures as a bank stabilization scheme is not known. Further research and monitoring of existing structures is needed to document the long-term performance and to develop more definitive design criteria.

6.1.3 VEGETATIVE METHODS FOR EROSION CONTROL

Vegetation is the basic component of bioengineering (Schiechtl, 1980) or biotechnical engineering (Gray and Leiser, 1982; Gray and Sotir, 1996). Schiechtl (1980) states that bioengineering requires *the skills of the engineer, the learning of the biologist, and the artistry of the landscape architect.* The concept of bioengineering is ancient, but there has been much recent research and documentation of the topic. The publications just cited, as well as Coppin and Richards (1990), provide comprehensive coverage, and many other works provide discussion of specialized aspects of the subject.

Vegetation can function as either armor or indirect protection, and in some applications, can function as both simultaneously. Grassy vegetation and the roots of brushy and woody vegetation function as armor, while brushy and woody vegetation function as indirect protection. The roots of vegetation may also add a degree of geotechnical stability to a bank slope through reinforcing the soil.

Some factors that affect the success of a bioengineering approach, such as weather and the timing and magnitude of streamflows, are beyond the control of the designer. Therefore, expert advice, careful planning, and attention to detail are critical to maximizing the probability of success.

Many streambank protection projects include vegetation without conscious thought by the designer, since native vegetation often establishes itself once the processes of bank failure are stopped by structural means. However, if the potential for utilizing vegetation is considered from the beginning, then the effectiveness, environmental aspects, and economy of a project can often be significantly improved.

Vegetation is most often used in conjunction with structural protection. Exceptions may be made for very small waterways, for areas of low erosion activity, or for situations where the consequences of failure are low and there is provision for rehabilitation in case of failure. Vegetation can have a particularly important role in the stabilization of upper bank slopes. Vegetation is especially appropriate for environmentally sensitive projects, whether benefits to recreation, esthetics, or wildlife is the object.

Vegetation is well-suited for incremental construction, either to wait for more favorable planting conditions for specific types of vegetation or to wait for deposition of sediments in the area to be planted. Vegetation is also suitable for inexpensive reinforcement or repair of existing erosion protection works in some situations.

Woody vegetation is useful in preventing or repairing scour at or behind top of bank, especially if the scour resulted from an infrequent flood event that is not likely to recur before the vegetation becomes effective.

Woody vegetation is sometimes used to prevent floating debris from exiting the channel during floods and becoming a nuisance in the floodplain.

Advantages: The two obvious advantages of vegetation as erosion protection are environmental attractions and relatively low cost. A third and less obvious attraction is that it can increase the safety factor of structural protection by enhancing the level of performance. Because many types of vegetative treatment are labor intensive, the cost advantage will be especially prominent in regions where labor is inexpensive, skilled in agriculture, and conscientious.

Disadvantages: Some characteristics which make vegetation effective and desirable in most situations may be disadvantages in other situations. However, many of the following concerns will either not be applicable for a specific project, or will be acceptable as compromises in light of vegetation's merits.

The most serious shortcoming is that even well executed vegetative protection cannot be planned and installed with the same degree of confidence, or with as high a safety factor, as structural protection. This is not to say that vegetation will not be adequate, or will not be more cost effective than structural protection in a specific situation, but is rather an acknowledgement that structural protection can be designed to function under more severe conditions of hydraulic and geotechnical instability than can vegetation. Vegetation is especially vulnerable to extremes of weather and inundation before it becomes well established.

Quantitative guidance for the use of vegetation in streambank protection is limited, although there has been progress through recent research. Most vegetative measures have constraints on the season of the year in which installation can be performed. This shortcoming can be mitigated to some degree by advance planning or by developing more than one option for vegetative treatment.

In arid regions, vegetation can reduce soil moisture which may be a concern. However, this is not likely to be a serious concern if the native plant ecosystem was considered in the initial selection of vegetative species.

Vegetative treatments often require significant maintenance and management to prevent the following problems:

Growth of vegetation causing a reduction in flood conveyance or causing erosive increases in velocity in adjacent unvegetated areas;

Trunks of woody vegetation or clumps of brushy vegetation on armor revetment causing local flow anomalies which may damage the armor;

Large trees threatening the integrity of structural protection by root invasion or by toppling and damaging the protection works, or by toppling and directing flow into an adjacent unprotected bank; and

Roots infiltrating and interfering with internal bank drainage systems, or causing excess infiltration of water into the bank.

Many of these problems may be avoided through selection of the appropriate type, and species of vegetation for the purpose. However, designers rarely have the practical experience or formal training in biotechnology to make such selections and expert advice must be obtained from qualified individuals in plant biology and bioengineering.

6.2 GRADE CONTROL

Implementation of bank stabilization measures without proper consideration of the stability of the bed can result in costly maintenance problems and failure of structures. The stability of the bed is an essential component of any channel stabilization scheme. Bank stabilization measures are generally

appropriate solutions to local instability problems; however, when system-wide channel degradation exists, a more comprehensive treatment plan must be implemented utilizing a combination of bank stabilization and grade control. Therefore, design of an effective grade control structure, or a system of structures, requires at a minimum, a common definition of stability, and a design procedure that relies on the balance between supply and transport of a desired sediment yield from the upstream watershed and channel system. As will be shown later in this chapter, reliance on emprical design relationships for grade control structures is self-defeating.

Equally important to the design fundamentals is an understanding of the functions of a grade control. In the broadest sense, the term *grade control* can be applied to any alteration in the watershed which provides stability to the streambed. The most common method of establishing grade control is the construction of in-channel grade control structures. There are basically two functions of grade control structures. One type of structure is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the degradational zone. This is somewhat analogous to locally increasing the size of the bed material. Lane's relation (Section 3.4.1) would illustrate the situation by $QS^{+} \propto Q_s D_{50}^{+}$, where the increased slope (S^{+}) of the degradational reach would be offset by an increase in the bed material size (D_{50}^{+}). For this discussion, this will be referred to as a *Bed Control Structure*. The other structure is designed to function by reducing the energy slope along the degradational zone ($QS^{-} \propto Q_s D_{50}$). This will be referred to as a *Hydraulic Control Structure*. The distinction between the processes by which these structures operate is important whenever grade control structures are considered.

Because of the complex hydraulic behavior of grade control structures, it is difficult to designate a single function that will apply without exception to each structure. For many situations, the function of a structure as either a bed control structure or hydraulic control structure is readily apparent. However, there may be circumstances where a single function of a structure as strictly a bed control or hydraulic control structure may be less evident and, in many cases, the structure may actually have characteristics of both. It also must be recognized that the hydraulic performance and, therefore, the function of the structure, can vary with time and discharge. This can occur within a single hydrograph or over a period of years as a result of upstream or downstream channel changes.

6.2.1 TYPES OF GRADE CONTROL STRUCTURES

There are certain features which are common to most grade control structures. These include a control section for accomplishing the grade change, a section for energy dissipation, and protection of the upstream and downstream approaches. However, there is considerable variation in the design of these features. For example, a grade control structure may be constructed of riprap, concrete, sheet piling, treated lumber, soil cement, gabions, compacted earth fill, or other locally available material. Also, the shape (sloping or vertical drop) and dimensions of the structure can vary significantly, as can the various appurtenances (baffle plates, end sills, etc.). The applicability of a particular type of structure to any given situation depends upon a number of factors such as: hydrologic conditions, sediment size and loading, channel morphology, floodplain and valley characteristics, availability of construction materials, project objectives, and time and funding constraints. The successful use of a particular type of structure in one

situation does not necessarily ensure it will be effective in another. Some of the more common types of grade control structures used in a variety of situations are discussed in the following sections. For more information on various structure designs, the reader is referred to Neilson *et al.* (1991), which provides a comprehensive international literature review on grade control structures with an annotated bibliography.

6.2.1.1 Simple Bed Control Structures

Perhaps the simplest form of a grade control structure consists of dumping rock, concrete rubble, or some other locally available non-erodible material across the channel to form a hard point. These structures are often referred to as rock sills, or bed sills. These type of structures are generally most effective in small stream applications and where the drop heights are generally less than about two to three 3 feet. A series of rock sills, each creating a head loss of about two feet was used successfully on the Gering Drain in Nebraska (Stufft, 1965). The design concept presented by Whitaker and Jaggi (1986) for stabilizing the streambed with a series of rock sills is shown in Figure 6.1. The sills in Figure 6.1 are classic bed control structures which are simply acting as hard points to resist the erosion of the streambed.

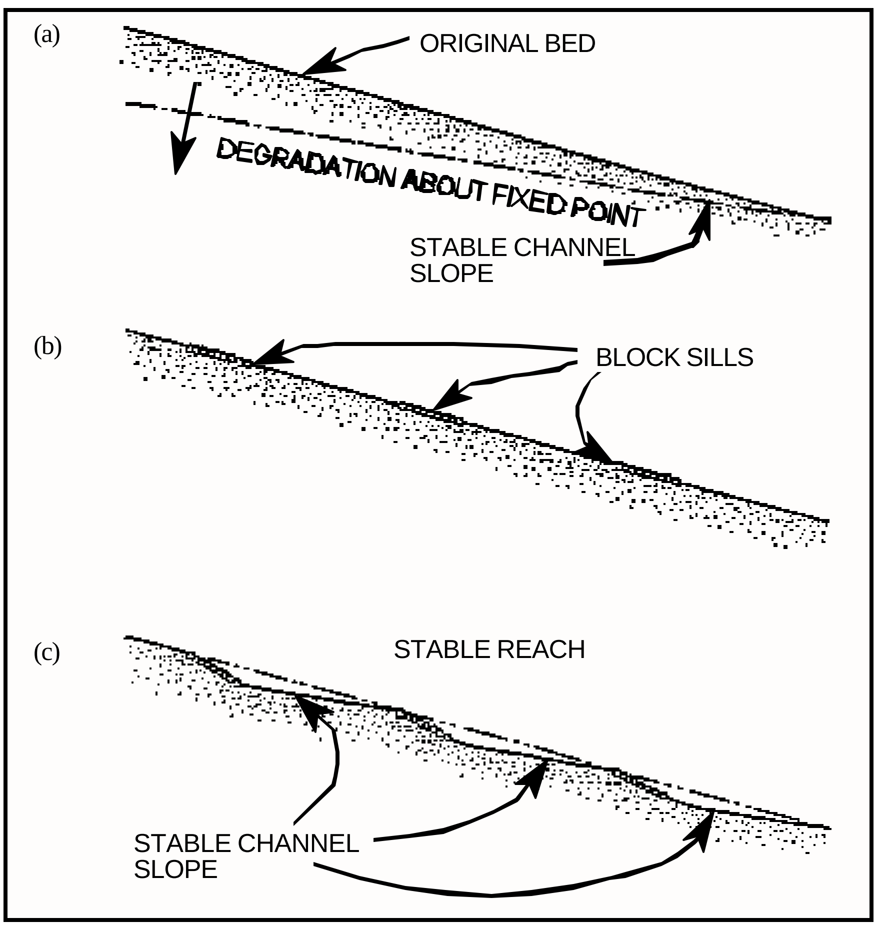


Figure 6.1 Channel Stabilization with Rock Sills (adapted from Whitaker and Jaggi, 1986)

Construction of bed sills is sometimes accomplished by simply placing the rock along the streambed to act as a hard point to resist the erosive forces of the degradational zone. In other situations, a trench may be excavated across the streambed and then filled with rock. A critical component in the design of these structures is ensuring that there is sufficient volume of non-erodible material to resist the general bed degradation, as well as the local scour at the structure. This is illustrated in Figures 6.2a and 6.2b which shows a riprap grade control structure designed to resist both the general bed degradation of the approaching knickpoint as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass to launch with an acceptable thickness to the anticipated scour hole depth.

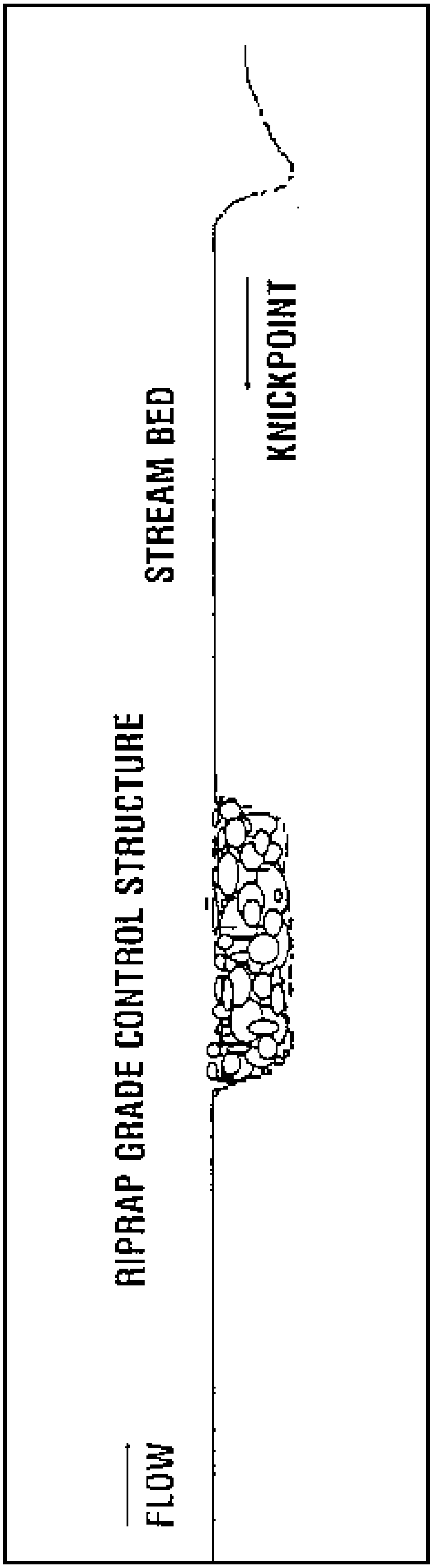
6.2.1.2 Structures with Water Cutoff

One problem often encountered with the above structures is the displacement of rock (or rubble, etc.) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the channel is composed primarily of pervious material. This problem can be eliminated by constructing a water barrier at the structure. One type of water barrier consists of simply placing a trench of impervious clay fill upstream of the weir crest. This type of water barrier is illustrated in Figures 6.3a and 6.3b. One problem with this type of barrier is its longevity due to susceptibility to erosion. This problem can be avoided by using concrete or sheet piling for the cutoff wall. The conceptual design of a riprap grade control structure with a sheet pile cutoff wall is shown in Figures 6.4a and 6.4b. In the case of the sloping riprap drop structures used by the Denver Urban Drainage and Flood Control District, an impervious clay fill is used in conjunction with a lateral cutoff wall (McLaughlin Water Engineers, Ltd., 1986). This design is illustrated in Figure 6.5.

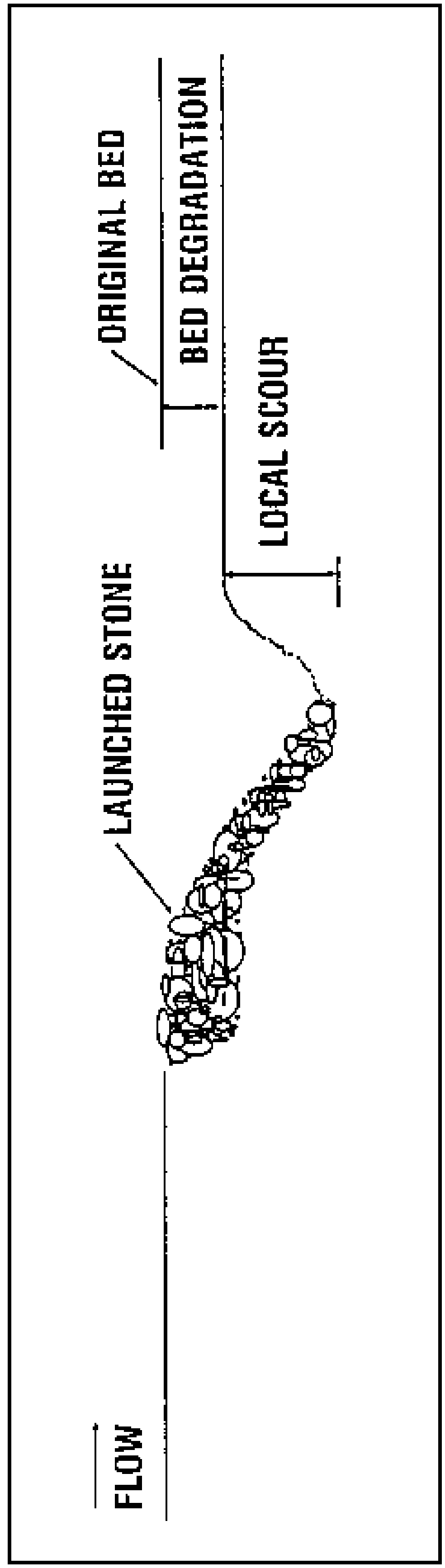
6.2.1.3 Structures with Pre-formed Scour Holes

A significant feature that distinguishes the sloping riprap structure of Figure 6.5 from the other structures is the preformed, rock protected scour hole. A scour hole is a natural occurrence downstream of any drop whether it is a natural overfall or a man-made structure. A rock grade control structure must have sufficient launching rock to protect against the vertical scour immediately downstream of the weir section. However, the lateral extent of the scour hole must also be considered to ensure that it does not become so large that the structure is subject to being flanked. With many simple grade control structures in small stream applications, very little, if any attention is given to the design of a stilling basin or pre-formed scour hole, but rather, the erosion is allowed to form the scour hole. However, at higher flow and drop situations, a pre-formed scour hole protected with concrete, riprap, or some other erosion resistant materials is usually warranted. This scour hole serves as a stilling basin for dissipating the energy of the plunging flow. Sizing of the scour hole is a critical element in the design process which is usually based on model studies or on experience with similar structures in the area.

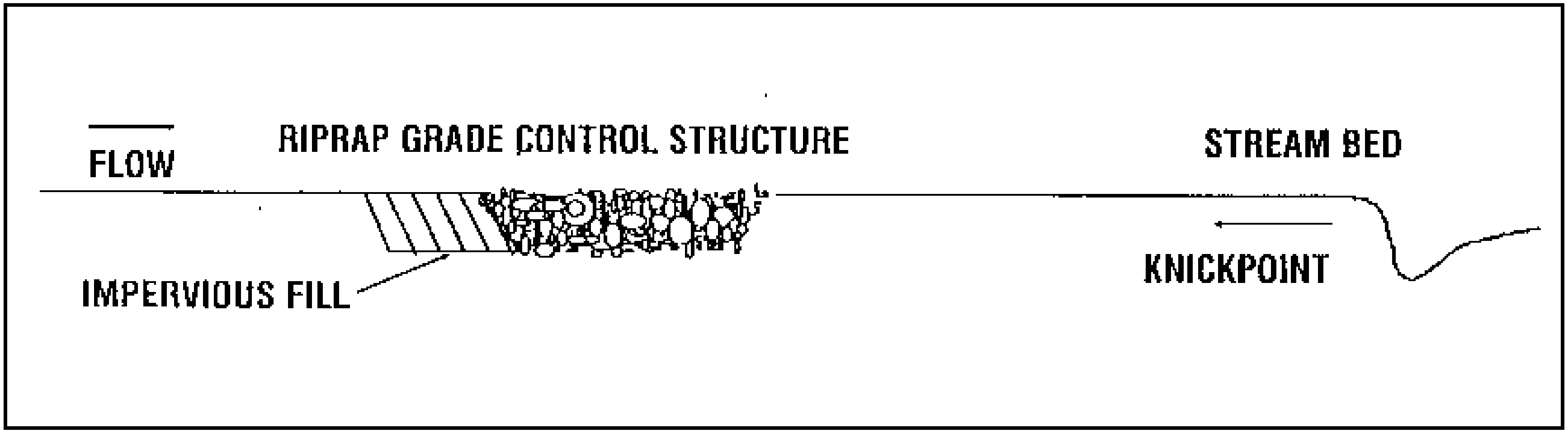
The stability of rock structures is often jeopardized at low tailwater conditions due to the stability of the rock, which is often the limiting factor in determining the maximum drop height of



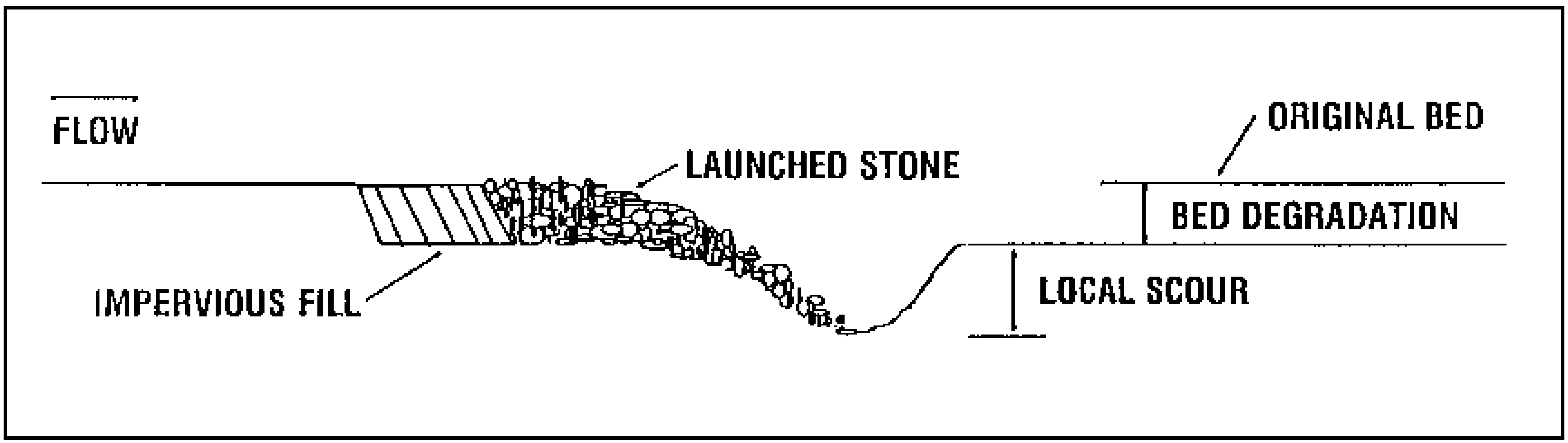
Figures 6.2a As Built Riprap Grade Control Structure with Sufficient Launch Stone to Handle Anticipated Scour



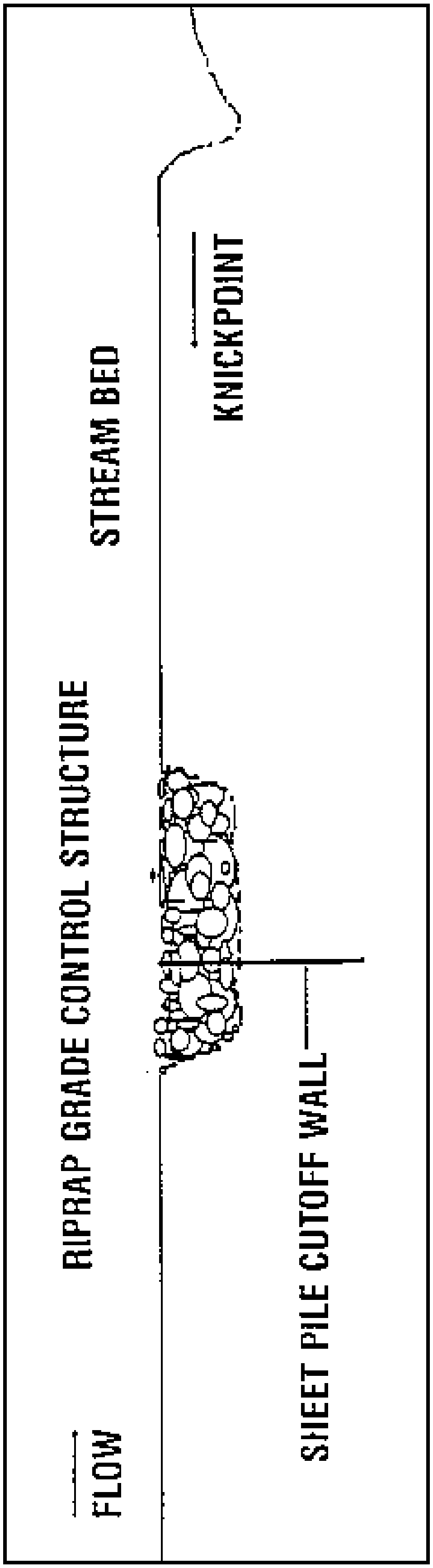
Figures 6.2b Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour



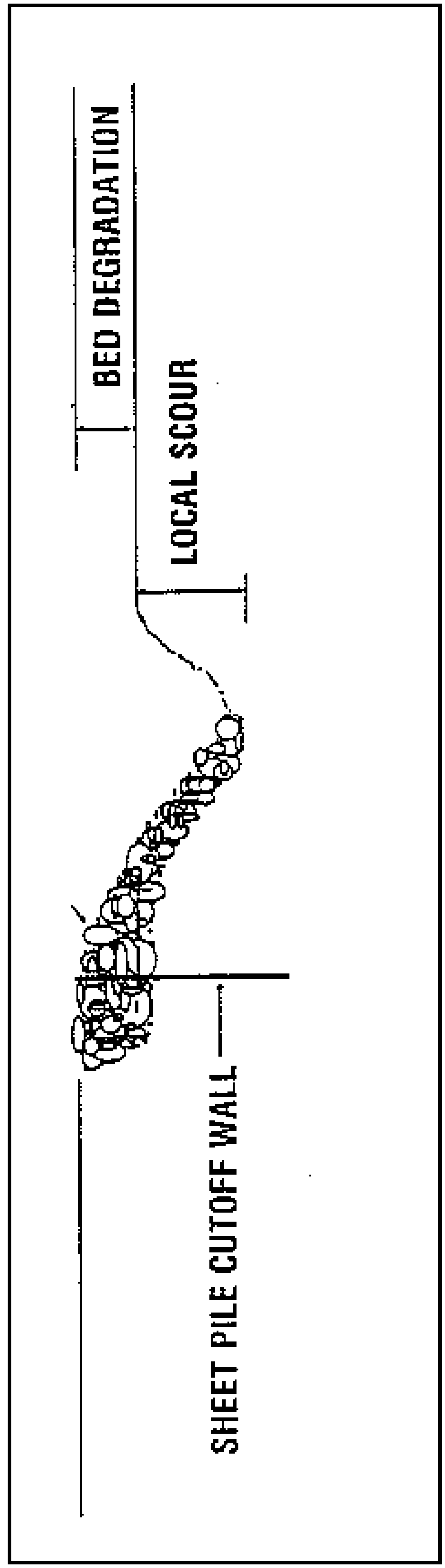
Figures 6.3a As Built Riprap Grade Control Structure with Impervious Fill Cutoff Wall



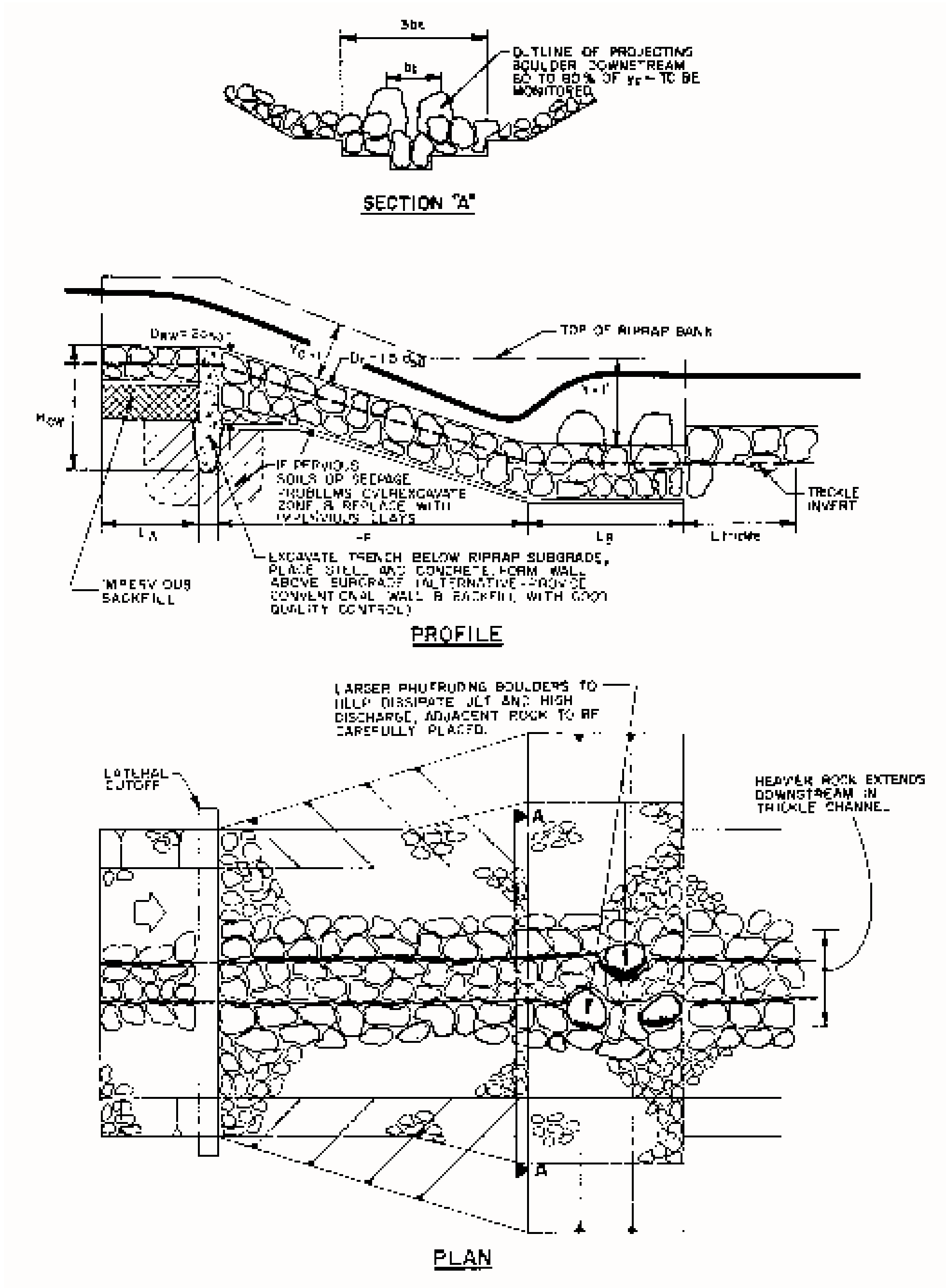
Figures 6.3b Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour



Figures 6.4a As Built Riprap Grade Control Structure with Sheet Pile Cutoff Wall



Figures 6.4b Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour



Figures 6.5 Sloping Drop Grade Control Structure with Pre-formed Riprap Lined Scour Hole (McLaughlin Water Engineers, 1986)

the structure. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. This is the basis for design of the bed stabilizer shown in Figure 6.6 (U.S. Army Corps of Engineers, 1970). These structures generally perform satisfactorily as long as they are designed to operate at submerged conditions where the tailwater (T') does not fall below 0.8 of the critical depth (D_c) at the crest section (Linder, 1963). Subsequent monitoring of the in place structures confirmed their successful performance in the field (U.S. Army Corps of Engineers, 1981).

In many instances, the energy dissipation in a grade control structure is accomplished by the plunging action of the flow into the riprap protected stilling basin. This is generally satisfactory where the degree of submergence is relatively high due to small drop heights and/or high tailwater conditions. However, at lower submergence conditions where drop heights are large or tailwater is low, some additional means of dissipating the energy must be provided. Little and Murphey (1982) observed that an undular hydraulic jump occurs when the incoming Froude number is less than 1.7. Consequently, Little and Murphey developed a grade control design that included an energy dissipating baffle to break up these undular waves (Figure 6.7). This structure, referred to as the ARS type low-drop structure, has been used successfully in North Mississippi for drop heights up to about six feet by both the U.S. Army Corps of Engineers and the Soil Conservation Service (U.S. Army Corps of Engineers, 1981). A recent modification to the ARS structure was developed following model studies at Colorado State University (Johns *et al.*, 1993; Abt *et al.*, 1994). The modified ARS structure, presented Figure 6.8 retains the baffle plate but adopts a vertical drop at the sheet pile rather than a sloping rock-fill section.

6.2.1.4 Concrete Drop Structures

In many situations where the discharges and/or drop heights are large, grade control structures are normally constructed of concrete. There are many different designs for concrete grade control structures. The two discussed herein are the California Institute of Technology (CIT) and the St. Anthony Falls (SAF) structures. Both of these structures were utilized on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and channel conditions (Stufft, 1965). Where the discharges were large and the channel depth was relatively shallow, the CIT type of drop structure was utilized. The CIT structure is generally applicable to low-drop situations where the ratio of the drop height to critical depth is less than one; however, for the Gering Drain project this ratio was extended up to 1.2. The original design of this structure was based on criteria developed by Vanoni and Pollack (1959). The structure was then modified by model studies at the WES in Vicksburg, Mississippi, and is shown in Figure 6.9, (Murphy, 1967). Where the channel was relatively deep and the discharges smaller, the SAF drop structure was used. This design was developed from model studies at the SAF Hydraulic Laboratory for the U.S. Soil Conservation Service (Blaisdell, 1948). This structure is shown in Figure 6.10. The SAF structure is capable of functioning in flow situations where the drop height to critical depth ratio is greater than one and can provide effective energy dissipation within a Froude number range of 1.7 to 17. Both the CIT and the SAF drop structures have performed satisfactorily on the Gering Drain for over 25 years.

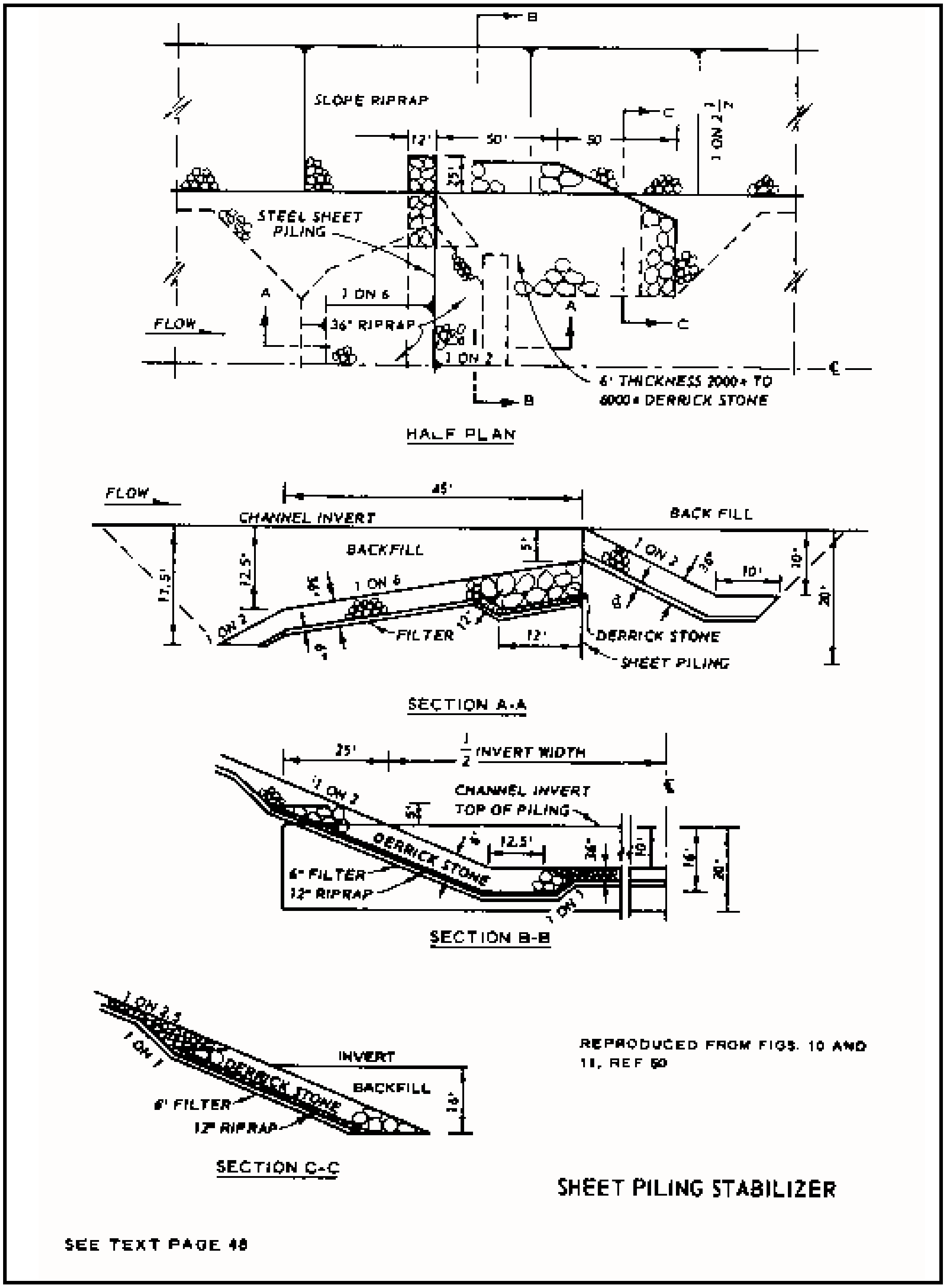


Figure 6.6 Bed Stabilizer Design with Sheet Pile Cutoff (U.S. Army Corps of Engineers, 1970)

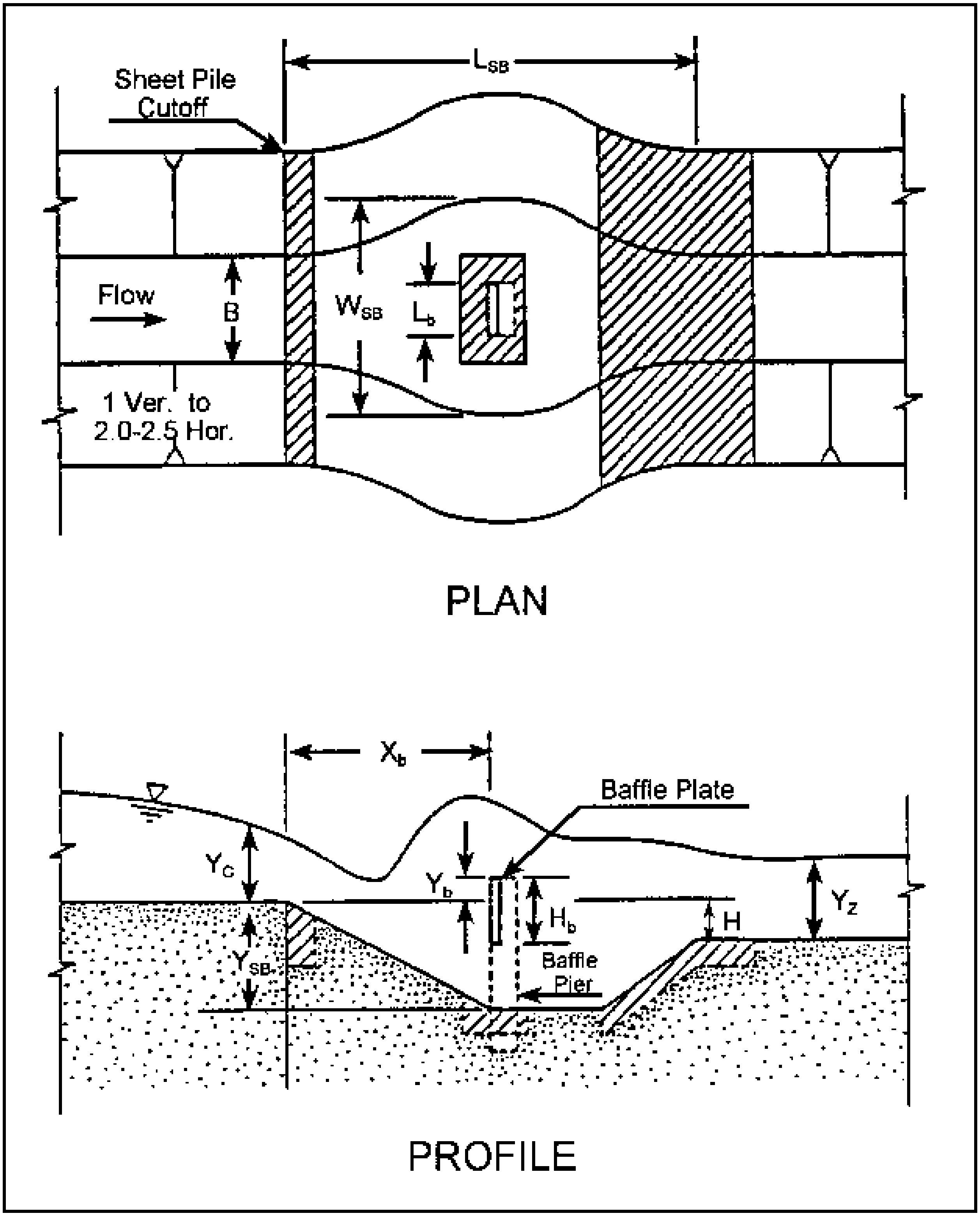


Figure 6.7 ARS-Type Grade Control Structure with Pre-formed Riprap Lined Stilling Basin and Baffle Plate (adapted from Little and Murphey, 1982)

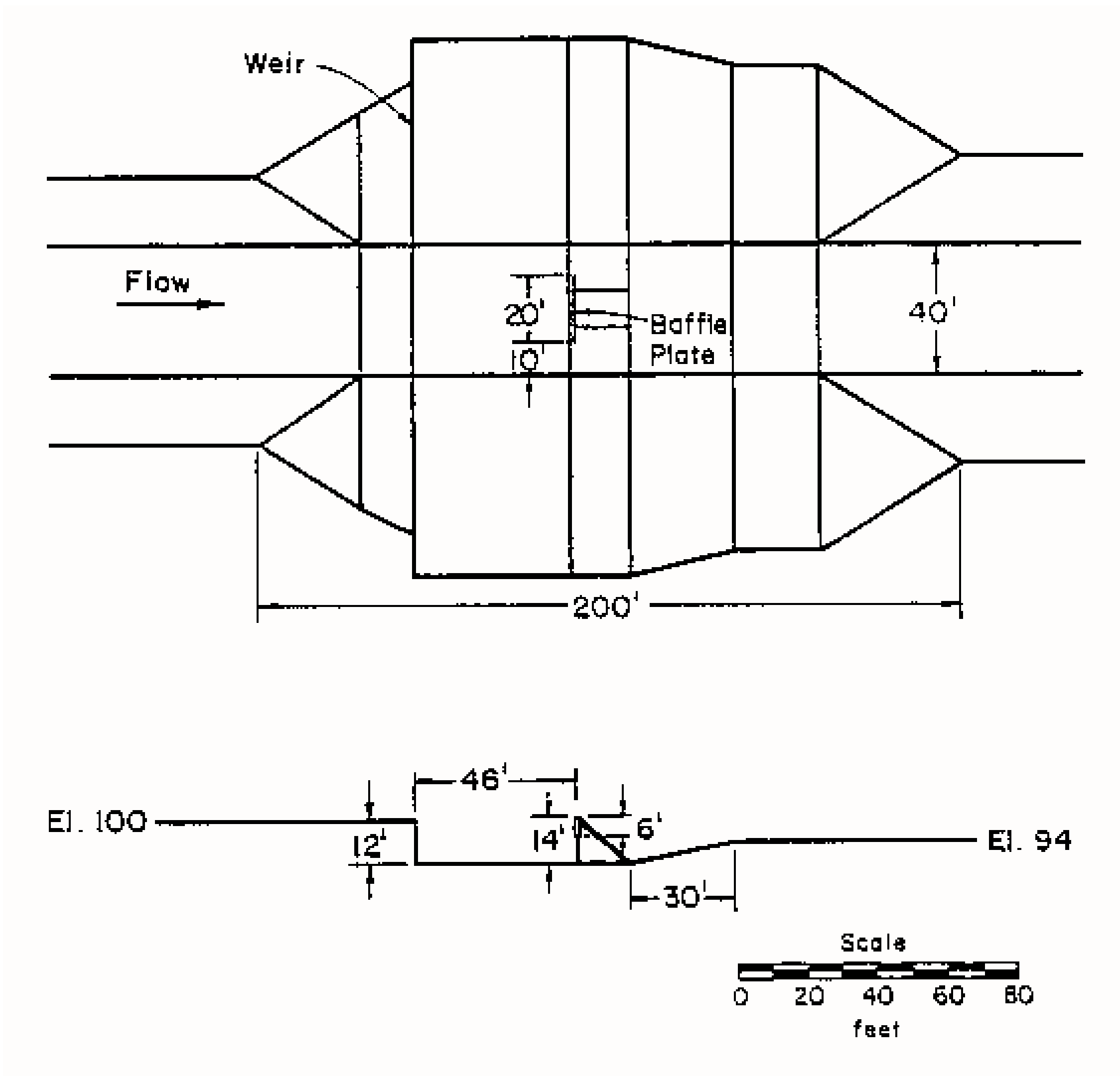


Figure 6.8 Schematic of Modified ARS-Type Grade Control Structure (Abt *et al.*, 1994)

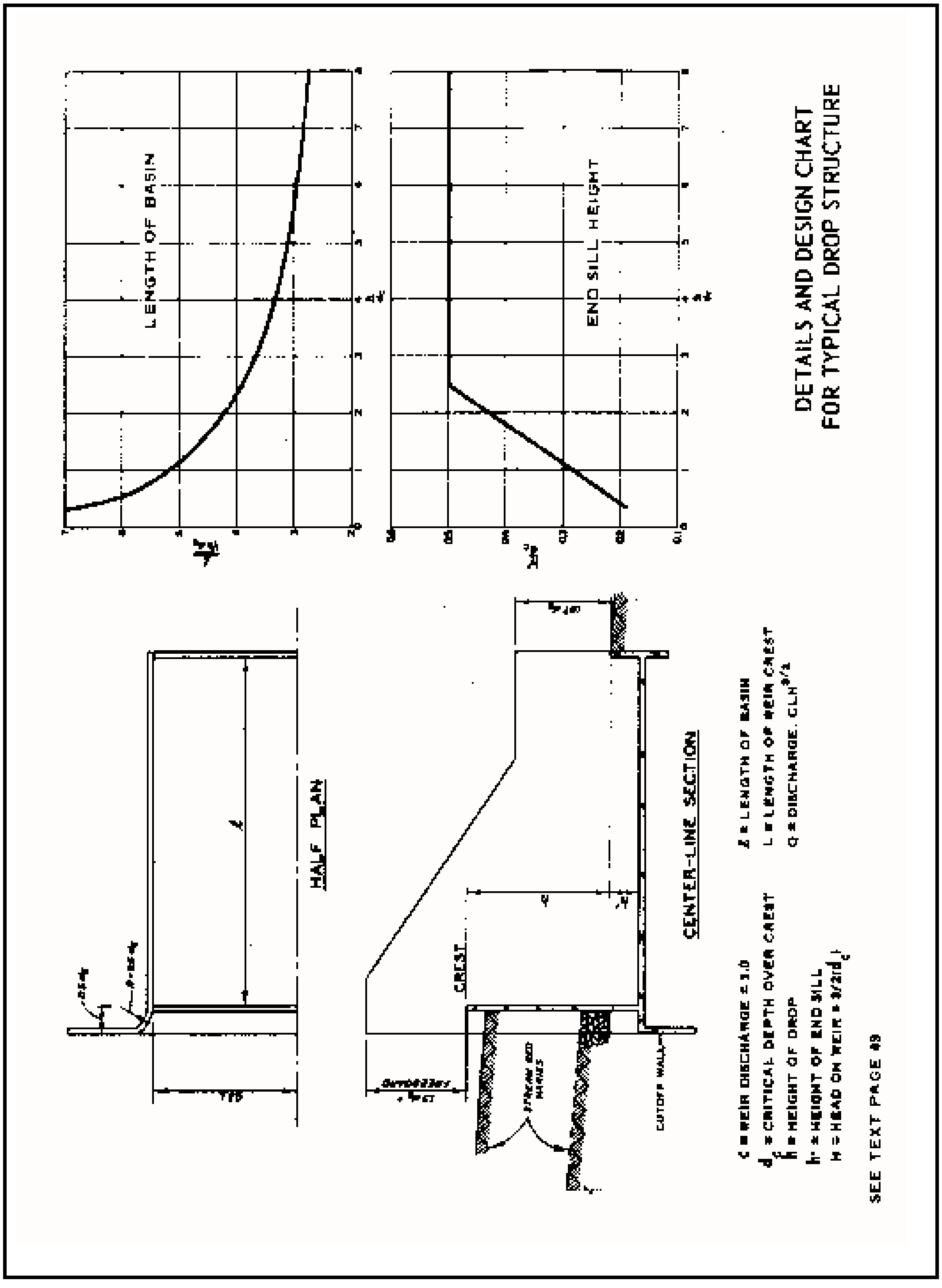


Figure 6.9 CIT-Type Drop Structure (Murphy, 1967)

6.2.1.5 Channel Linings

Grade control can also be accomplished by lining the channel bed with a non-erodible material. These structures are designed to ensure that the drop is accomplished over a specified reach of the channel which has been lined with riprap or some other non-erodible material. Rock riprap gradient control structures have been used by the U.S. Soil Conservation Service for several years (U.S. Soil Conservation Service, 1976). These structures are designed to flow in the subcritical regime with a constant specific energy at the design discharge which is equal to the specific energy of flow immediately upstream of the structure (Myers, 1982). Although these structures have generally been successful, there have been some associated local scour problems. This precipitated a series of model studies at the WES to correct these problems and to develop a design methodology for these structures (Tate, 1988 and 1991). A plan and profile drawing of the improved structure is shown in Figure 6.11.

6.2.1.6 Alternative Construction Materials

While riprap and concrete may be the most commonly used construction materials for grade control structures, there are many situations where cost or availability of materials may prompt the engineer to consider other alternatives. Gabion grade control structures are often an effective alternative to the standard riprap or concrete structures (Hanson *et al.*, 1986). Guidance for the construction of gabion weirs is also provided by the USACE (1974).

Another alternative to the conventional riprap or concrete structure which has gained popularity in the southwestern U.S. is the use of soil cement grade control structures. These structures are constructed of on site soil-sand in a mix with Portland Cement to form a high quality, erosion resistant mixture. Soil cement grade control structures are most applicable when used as a series of small drops in lieu of a single large-drop structure. Experience has indicated that a limiting drop height for these structures is on the order of three feet. Design criteria for these structures is presented by Simons, Li, & Associates, Inc. (1982).

6.2.2 EFFECTIVENESS OF GRADE CONTROL STRUCTURES

Design considerations for improving the effectiveness of grade control structures include determination of the type, location and spacing of structures along the stream, along with the elevation and dimensions of structures. Siting grade control structures is often considered a simple optimization of hydraulics and economics. However, hydraulics and economics alone are usually not sufficient to define the optimum spacing for grade control structures. In practice, the hydraulic considerations must be integrated with a host of other factors, which vary from site to site, to determine the final structure plan. Each of these factors should be considered in determining the effectiveness of the structures.

One of the most important steps in the siting of a grade control structure or a series of structures is the determination of the anticipated drop at the structure. This requires some knowledge

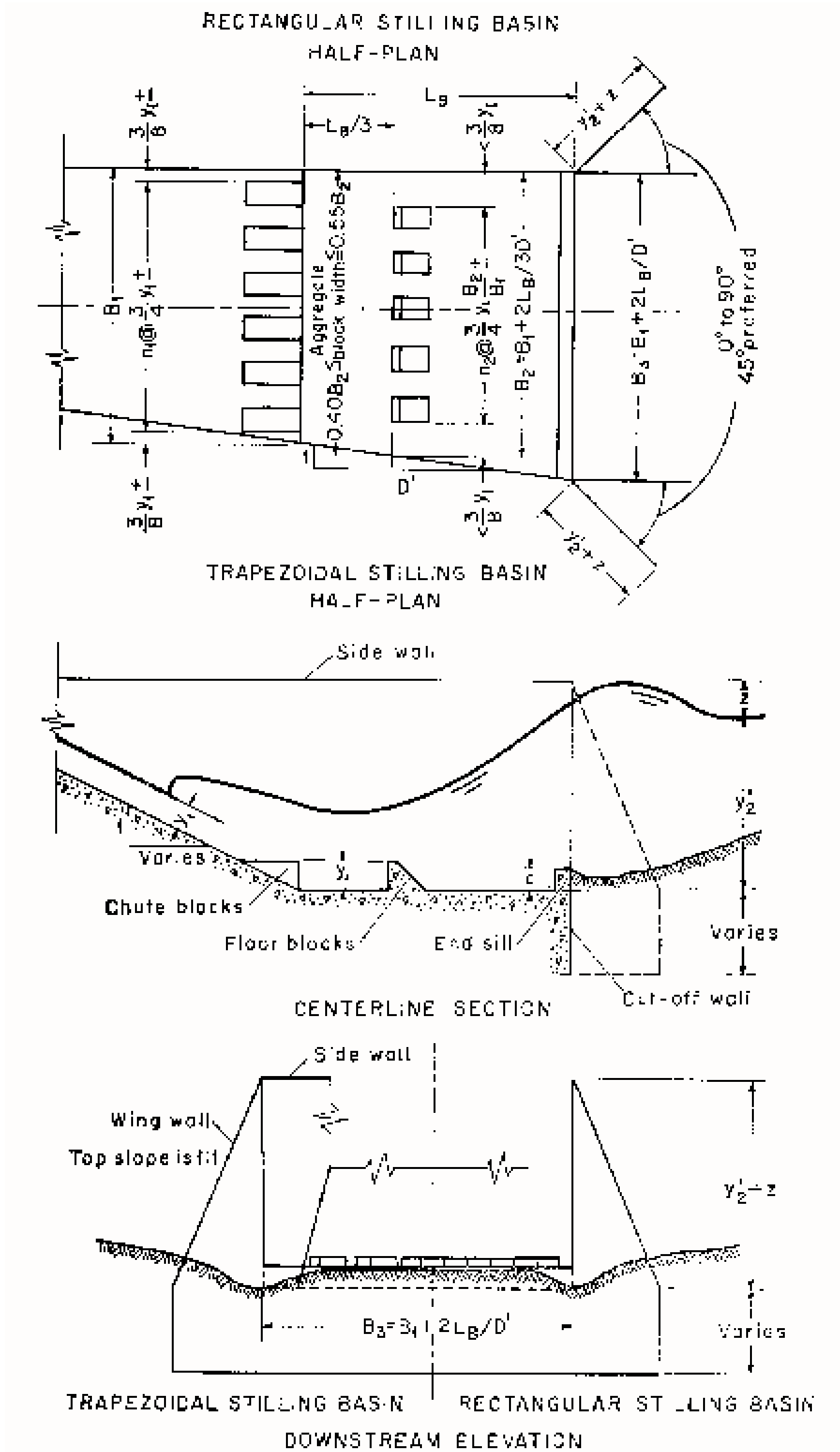


Figure 6.10 St. Anthony Falls (SAF) Type Drop Structure (Blaisdell, 1948)

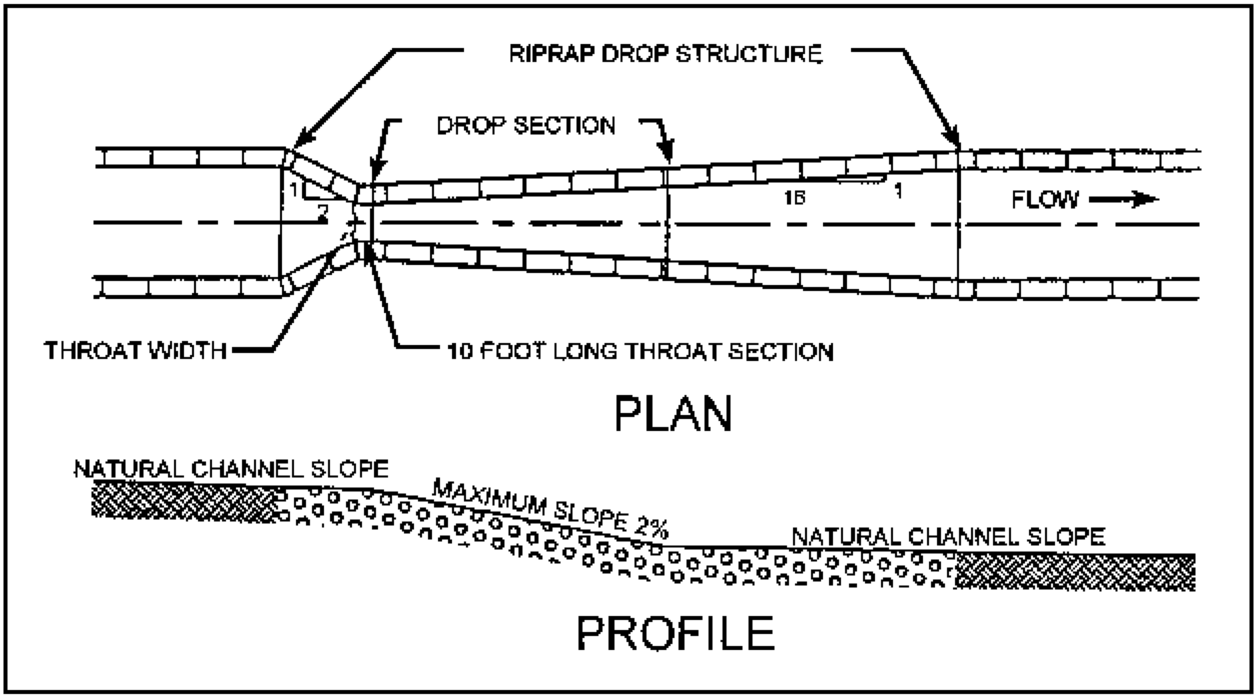


Figure 6.11 Riprap Lined Drop Structures (adapted from Tate, 1991)

of the ultimate channel morphology, both upstream and downstream of the structure which involves assessment of sediment transport and channel morphologic processes.

The hydraulic spacing of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation which, in turn, is a function of the next structure downstream. Heede and Mulich (1973) suggested that the optimum spacing of structures is such that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (1982) showed that the optimum spacing should be the length of the deposition above the structure that is a function of the deposition slope (Figure 6.12). Figure 6.12 also illustrates the recommendations of Johnson and Minaker (1944), that the most desirable spacing can be determined by extending a line from the top of the first structure at a slope equal to the maximum equilibrium slope of sediment upstream until it intersects the original streambed. However, each of the above references implicitly includes a specific sediment supply concentration, and that concentration is necessary for rational designs.

Theoretically, the hydraulic spacing of grade control structures is straightforward and can be determined by:

$$H = (S_o - S_f)x \tag{6.2}$$

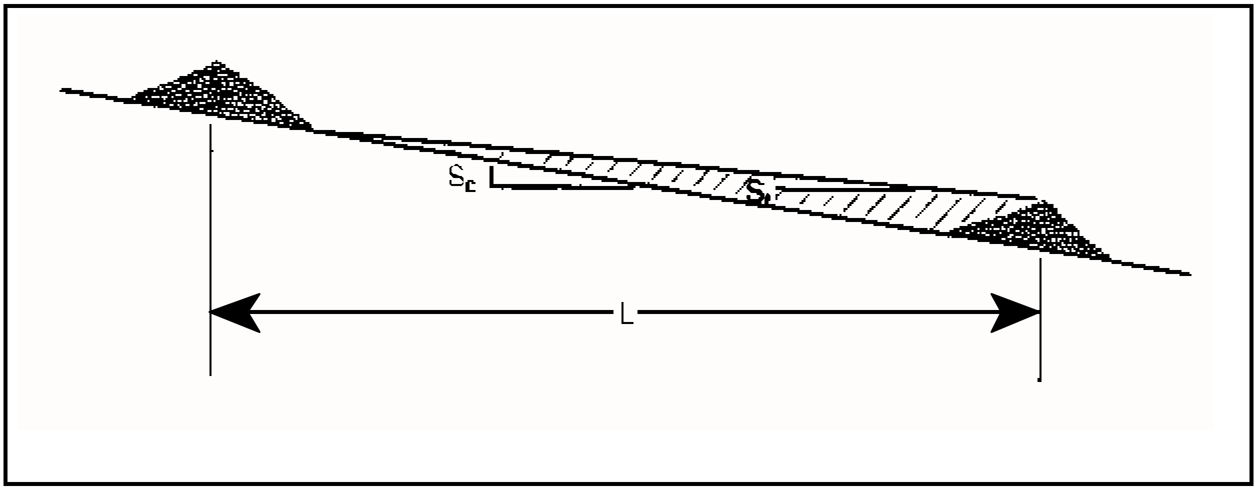


Figure 6.12 Spacing of Grade Control Structure (adapted from Mussetter, 1982)

where: H = the amount of drop to be removed from the reach;
S_o = the original bed slope;
S_f = the final, or equilibrium slope; and
x = the length of the reach (Goitom and Zeller, 1989).

The number of structures (N) required for a given reach can then be determined by:

$$N = H/h \tag{6.3}$$

where h is the selected drop height of the structure.

It follows from Eq. (6.2) that one of the most important factors when siting grade control structures is the determination of the equilibrium slope (S_f). Failure to properly define the equilibrium slope can lead to costly, overly conservative designs, or inadequate design resulting in continued maintenance problems and possible complete failure of the structures. Clearly, equilibrium slope (S_f) is a function of the sediment supply and is the slope required to transport the sediment supplied.

A critical element to designing for long-term sediment yield reduction is to explicitly include sediment transport and sediment yield in the design process. The USACE General Design Memorandum (GDM) No. 54 (USACE, 1990a) primarily uses a regional stability curve to design the spacing and height of grade control structures. The regional stability curve presented as Figure 6.13 is a relationship between thalweg slope and drainage area, and was developed by plotting the slope and drainage area of stable channel reaches. Figure 6.13 depicts the original data, the regression of the original data, and data from the 1995 monitoring of stream reaches. Stability was generally defined in terms of the Channel Evolution Model (CEM) (Schumm *et al.*, 1984) as discussed in Section 3.2. Regression of the original data used in GDM No. 54 results in the following relationship:

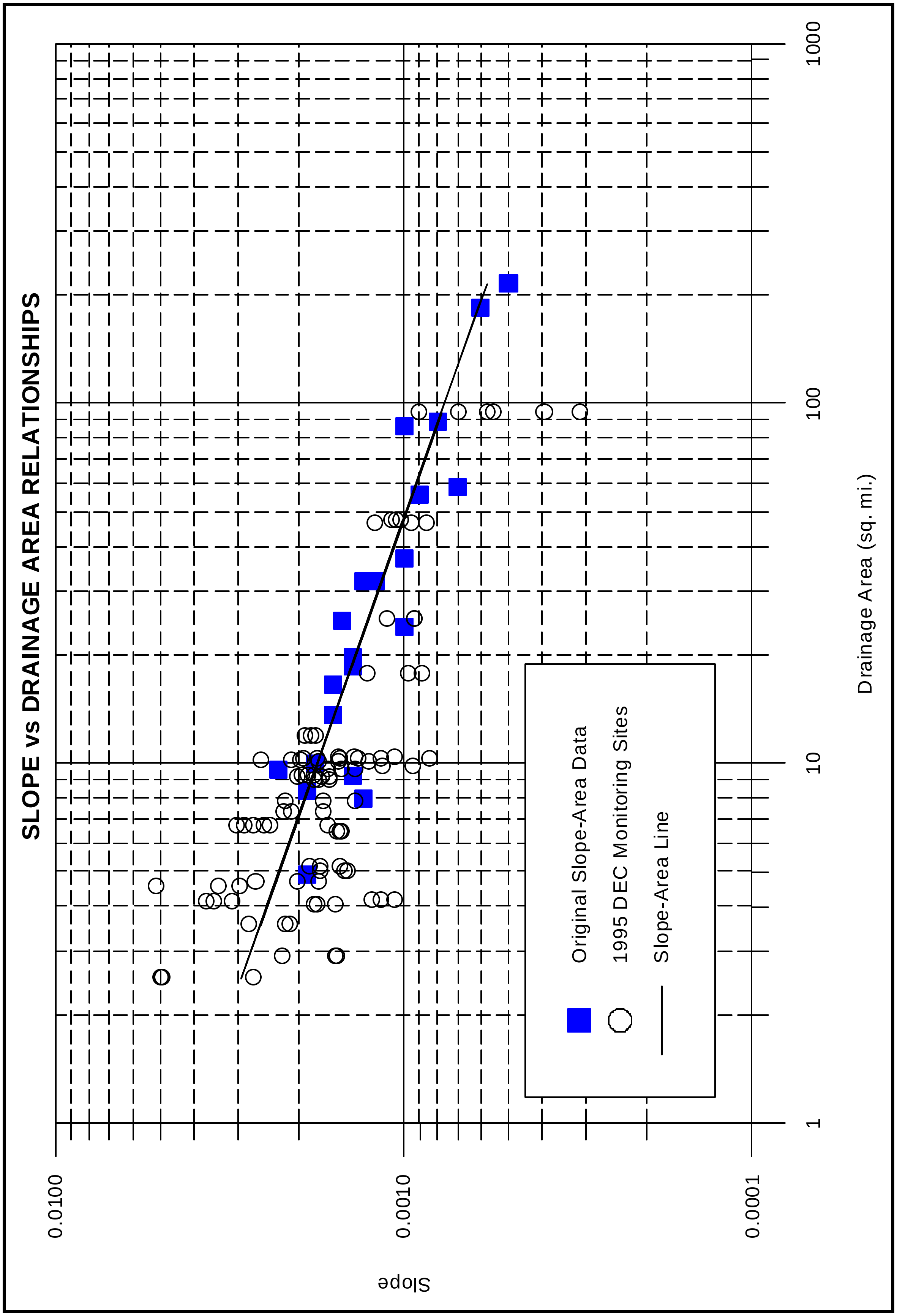


Figure 6.13 Slope Versus Drainage Area Relationship

$$S = 0.0041 \times A^{-0.365} \tag{6.4}$$

where: S = the stable slope; and
A = the drainage area in square miles.

One factor to consider is the drainage area size in developing this type of empirical relationship. As shown in Figure 6.13, only one reach less than 8 square miles was included in the original data, whereas, a later sample of watersheds in the same vicinity are primarily in the range of 2 to 10 square miles. Figure 6.14 is a comparison of the DEC monitoring reach energy slope data shown as CEM types, and the GDM No. 54 slope-area curve. For the portion of the slope-area curve greater than 10 square miles, most of the reaches are CEM 4 or CEM 5, indicating a reasonable degree of stability. For drainage areas less than 10 square miles, the slope-area curve is defined by CEM 2 or CEM 3, generally unstable reaches. The CEM 4 data less than 10 square miles in drainage area are below the regression relationship (Eq. 6.4).

Figure 6.15 is similar to Figure 6.13, with the following exceptions: a) 1995 DEC monitoring reach data for only CEM 4 and CEM 5 reaches are plotted; and b) these data exclude reaches that are ponded as a result of grade control construction. Ponding was not included in the original conception of the CEM. A new regression was made of the plotted data and the following relationship was plotted using a solid line (Figure 6.15):

$$S = 0.0018 \times A^{-0.145} \tag{6.5}$$

Parameters are as previously noted. The GDM No. 54 relationship is shown above as the dash line. The primary reasons for lowering and flattening of the relationship is that the sediment supply to the reaches has been reduced by upstream grade control structures and other measures emplaced by the DEC Project. Therefore, the prior empirical relationship (Eq. 6.4) is now invalid due to the effectiveness of erosion control measures.

Stability, as defined by the CEM criteria, includes a balance between sediment supply and sediment transport capacity. As the sediment supply has been reduced, the stable slope must also be reduced. Therefore, although the slope-area curve is a useful benchmark for comparison of reaches, the curve will require updating as success occurs in reducing sediment supply. Consideration should be given to using design procedures that explicitly include sediment supply and transport capacity.

Unfortunately the empirical slope-area regional stability curve, although useful, does not explicitly include sediment yield or sediment transport capacity. The relationships only implicitly include the sediment yield of the stable channels used in the data base. Figure 6.16 depicts the relationship between the energy slope and the computed sediment concentration in the DEC monitoring reaches. A regression expression for the sediment concentration data is:

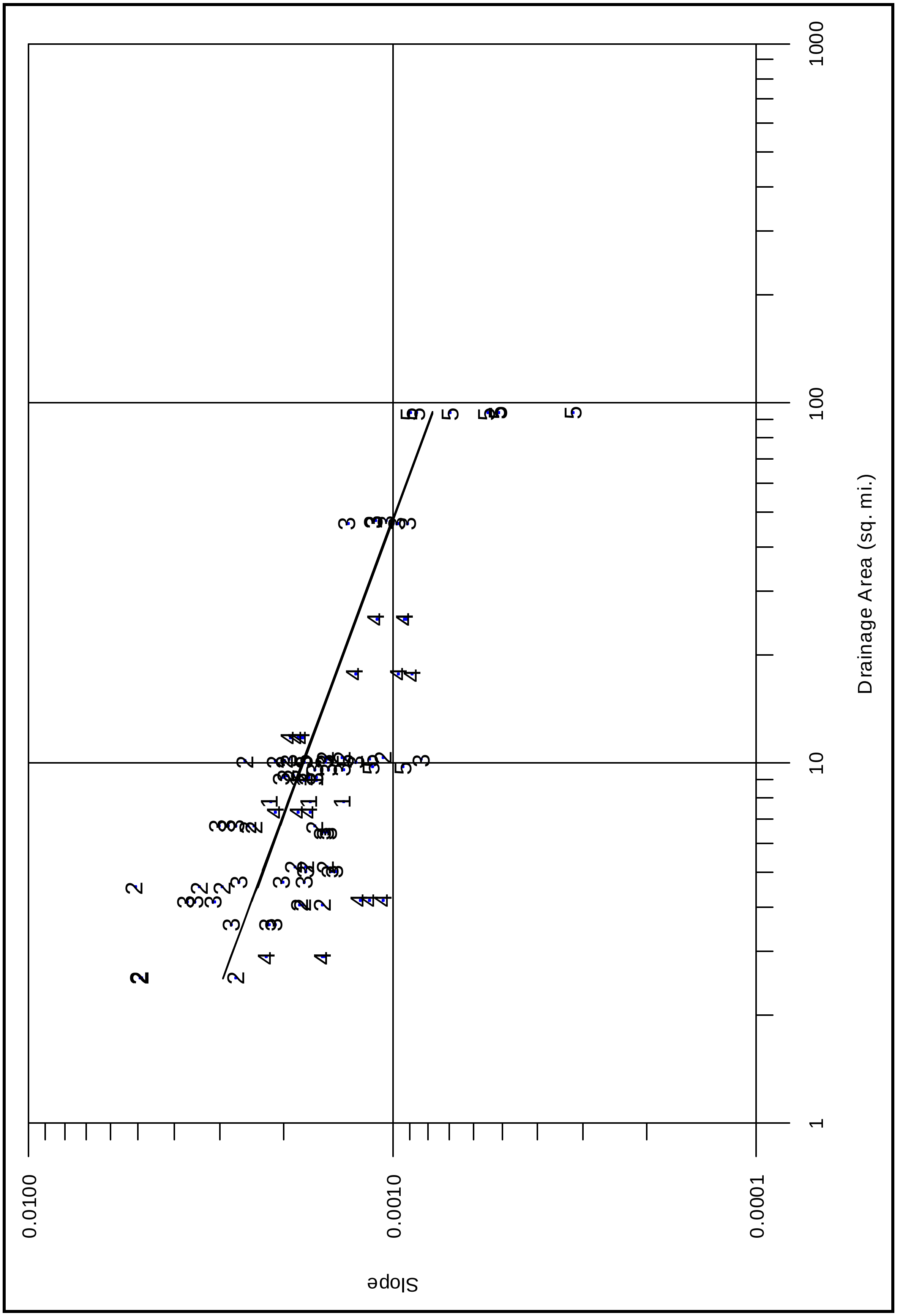


Figure 6.14 CEM Types in Comparison With a Slope Area Curve

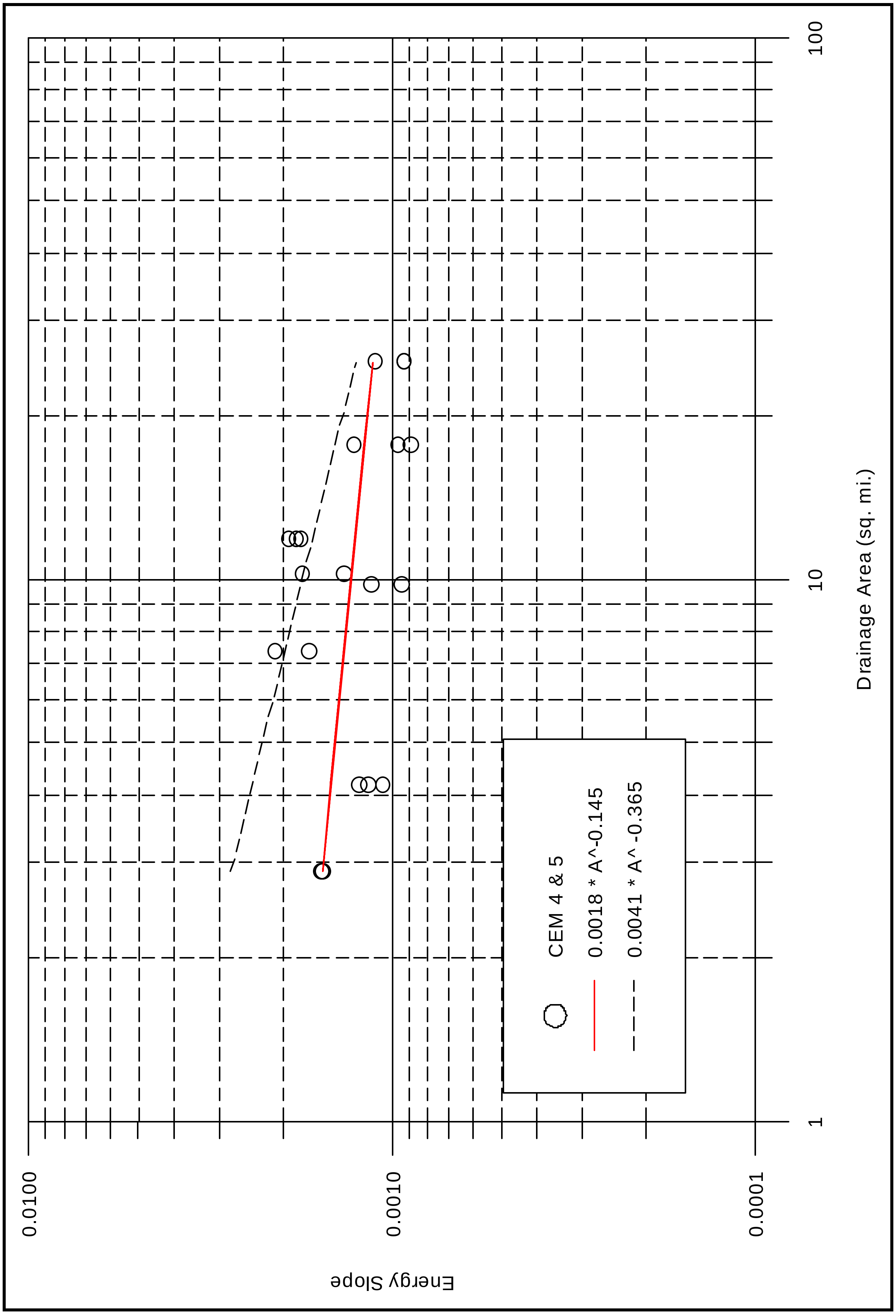


Figure 6.15 1995 CEM Data With Two Regressions

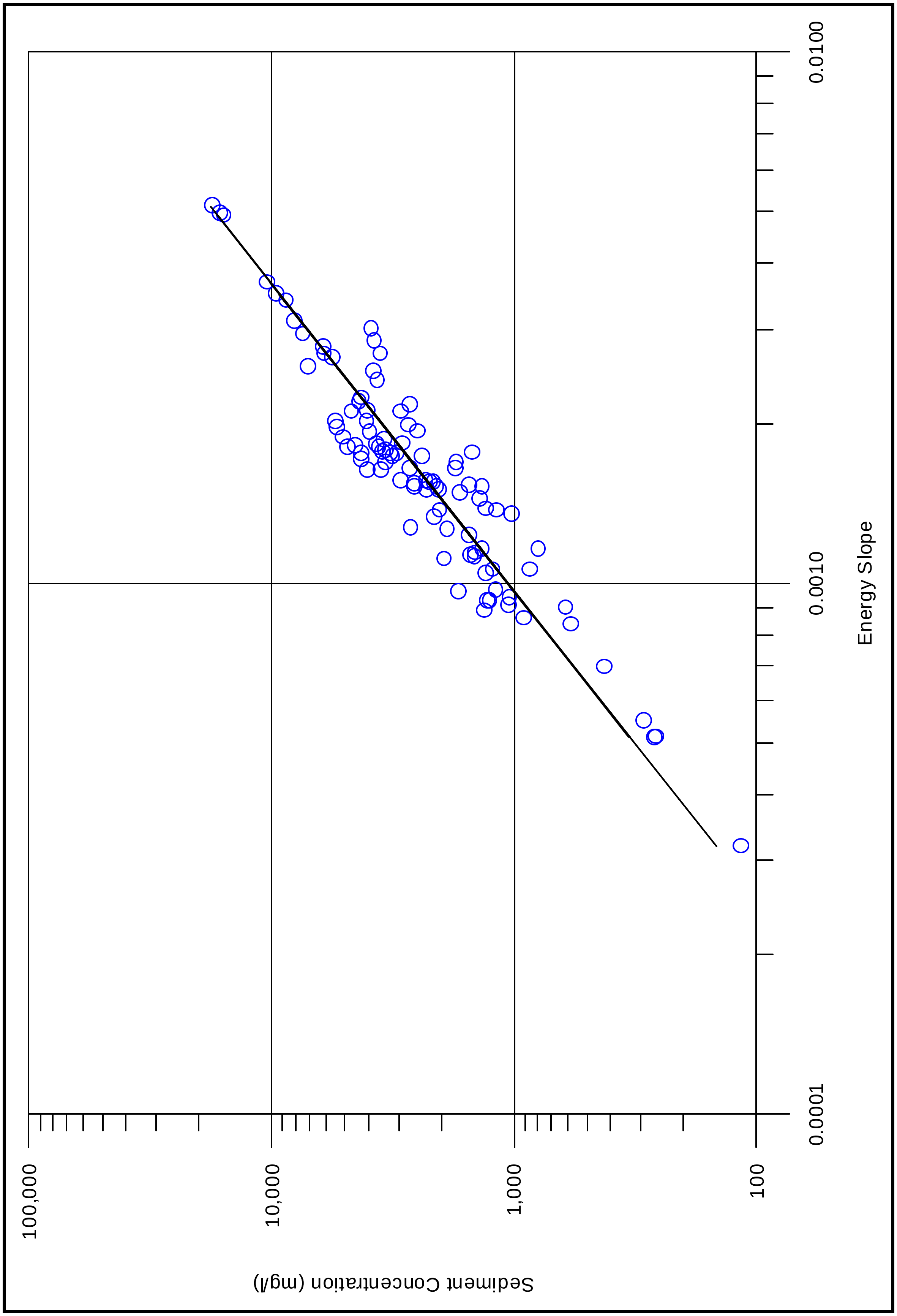


Figure 6.16 Relationship Between Energy Slope and Computed Sediment Concentration for DEC Monitoring Reaches

$$\text{Concentration} = 164{,}104{,}428 \times S^{1.73} \tag{6.6}$$

with the units of concentration as mg/l, and S is the energy slope. The coefficient of determination (R^2) is 0.89. As shown in Figure 6.16, the sediment concentration for S = 0.0009 is approximately 1000 mg/l, while for S = 0.004 the concentration is approximately 10,000 mg/l. Figure 6.17 shows the slope-area curve from GDM No. 54, and has values of sediment concentration taken from Figure 6.16 for selected drainage areas. Therefore, using the slope-area curve for stable channel design would require the designer to accept 712 mg/l at 90 square miles, 2,849 mg/l at 10 square miles, and extrapolating the relationship, 7,870 mg/l at 2 square miles. Therefore, investigation of an empirical spacing procedure has shown the effectiveness of grade control structures in northern Mississippi.

6.2.2.1 Downstream Channel Response

Since grade control structures affect the sediment delivery to downstream reaches, it is necessary to consider the potential impacts to the downstream channel when grade control structures are planned. Bed control structures reduce the downstream sediment loading by preventing the erosion of the bed and banks, while hydraulic control structures have the added effect of trapping sediments. The ultimate response of the channel to the reduction in sediment supply will vary from site to site. In some instances the effects of grade control structures on sediment loading may be so small that downstream degradational problems may not be encountered. However, in some situations such as when a series of hydraulic control structures is planned, the cumulative effects of sediment trapping may become significant. In these instances, it may be necessary to modify the plan to reduce the amount of sediment being trapped or to consider placing additional grade control structures in the downstream reach to protect against the induced degradation. Therefore, following the hydraulic spacing of a series of grade control structures using a thorough investigation of providing a balance between supply and transport of water and sediment, the designer must utilize a long-term sediment routing model such as HEC-6 (Thomas, 1996), to investigate downstream channel response.

An improved structure spacing procedure would be to select an energy slope based on the desired sediment transport concentration. The sediment transport concentration of the CEM 5 reaches within the DEC monitoring reaches can be used to select a design slope. Figure 6.18 provides a summary of the sediment concentration for CEM types for 1993, 1994 and 1995; the line through the data is the average for each CEM type. Figure 6.16 can be used to estimate the energy slope. The data indicates the design slope for the CEM 5 concentration of 1,000 mg/l would be approximately 0.001, and the CEM 4 concentration of 2,000 mg/l would be 0.0014. Structures could be located using this range of bed slopes, which would reduce sediment concentration below the existing average sediment transport. A check could then be made comparing bed slope and energy slope, and adjustments could be made if required.

The proposed optional procedure has the limitation of depending on the present field identification of CEM 4 and CEM 5 reaches. Just as with the GDM No. 54 slope-area curve, as the

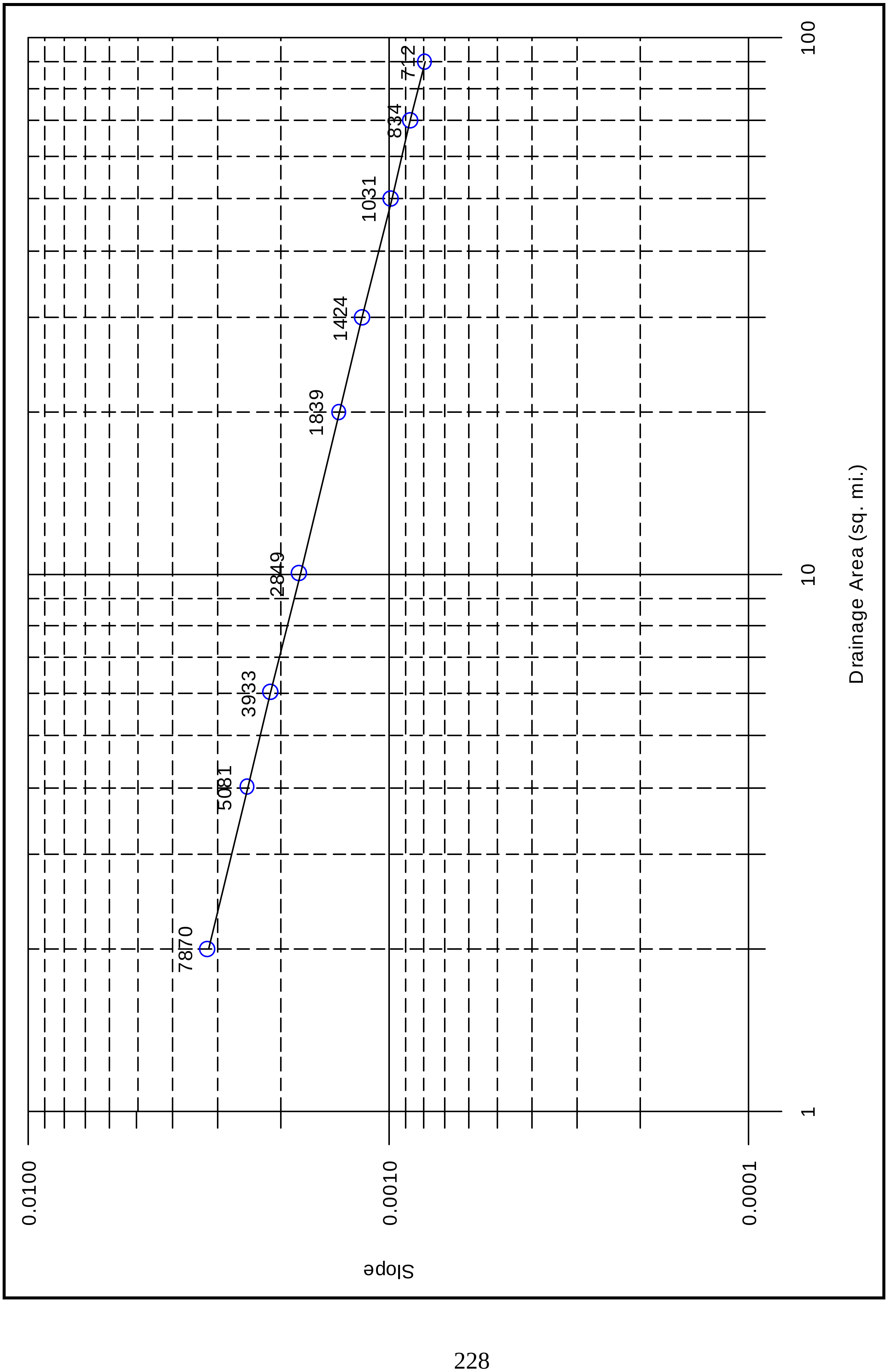


Figure 6.17 Sediment Concentration for the 2-Year Discharge Along the GDM No. 54 Slope Area Curve

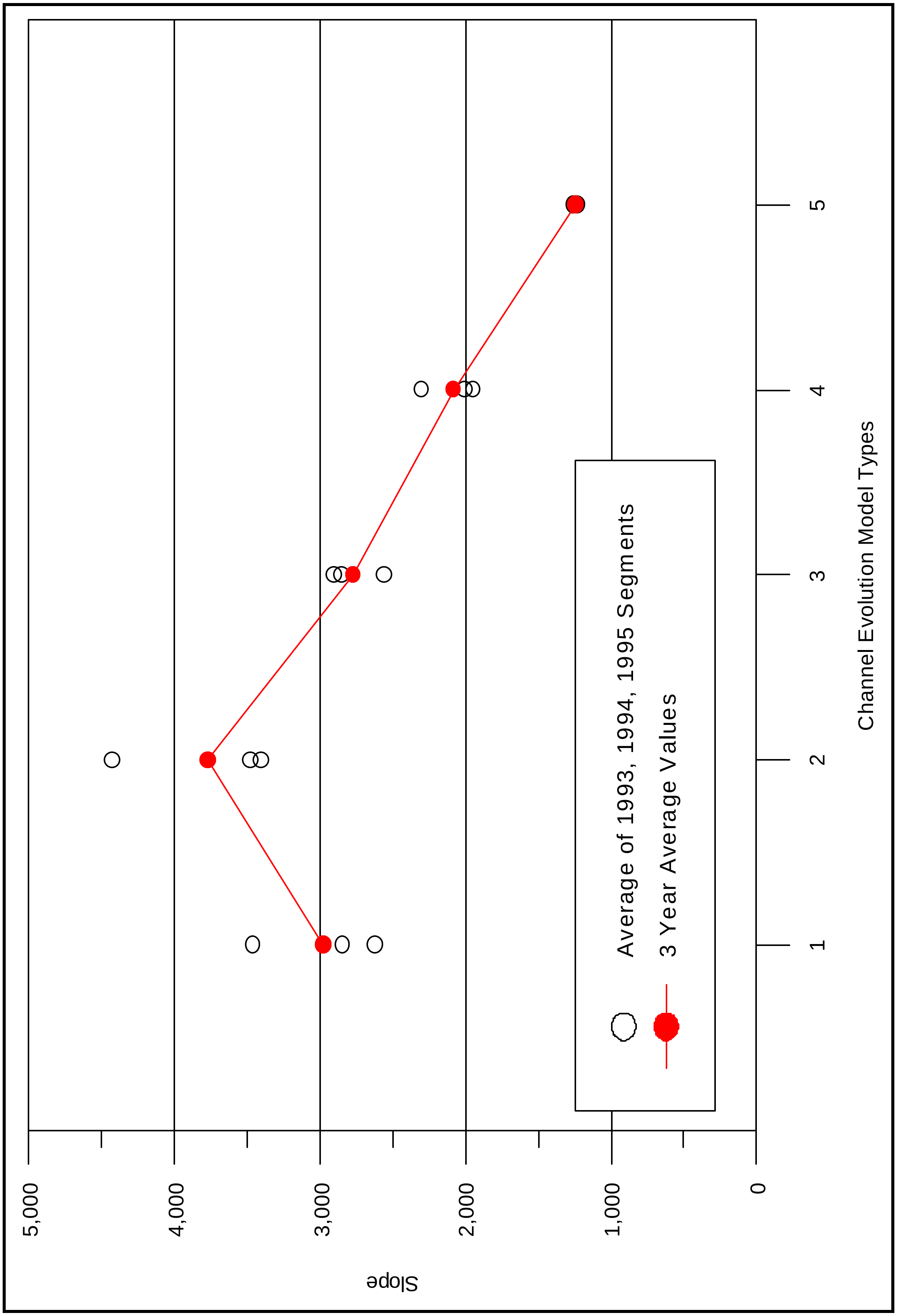


Figure 6.18 Computed Sediment Concentration for CEM Types

watersheds continue to stabilize the sediment concentrations will decrease, requiring that the sediment concentration of the CEM 4 and CEM 5 reaches be reviewed and, perhaps updated.

A rational approach to selecting the proper sediment supply would be approached in the following steps:

1. Assess sediment sources such as gullies, bank erosion, overbank watershed sources, and others to estimate the total watershed sediment yield on an annual basis using comparative surveyed cross sections, aerial photography, watershed models, etc.; and

2. From that assessment, estimate the sediment sources that could be eliminated using drop pipes, bank stabilization, grade control, and land use management practices to determine a best-practice sediment supply for the watershed.

The sediment transport capacity of the channel reach would then be computed using the following steps:

3. Develop a sediment rating curve similar to Figure 6.16 either from measured data if available or from sediment transport computations;

4. Generate a flow-duration curve, i.e., a relationship between the discharge and the percentage of time during the year that a particular discharge occurs;

5. Compute the annual sediment yield as the summation of products of the rating curve and the flow-duration curve; and

6. Adjust either the sediment rating curve using grade control, or the flow-duration curve using reservoir detention to meet the best-practice sediment supply for the watershed.

Standard computational procedures could then be used to check steady discharge or long-term simulation of the channel response. This proposed procedure is more intensive; however, additional planning elements and solution methods could be considered, and detail analysis is generally only a small fraction of construction costs for watershed stabilization.

Comparison of the annual yield from the DEC survey reaches (Table 6.1) indicates that sediment yield has decreased 20%, as expressed as an average of all reaches. Segregating those reaches with and without grade control structures indicates that those reaches with grade control structures have decreased sediment yield by 36%, while reaches without grade control have increased sediment yield by an average of 8%, compared to 1993 levels. Long-term records are required before definitive conclusions can be made regarding effectiveness of these structures.

Table 6.1 Annual Sediment Yield for Monitoring Reaches Surveyed in 1993, 1994, 1995, and 1996

		ANNUAL SEDIMENT YIELD (cubic yards)			
Site	Reach No.	1993	1994	1995	1996
Abiaca 6	1	15,369	24,482	20,972	1,319
Fannegusha	1	42,529	45,722	32,709	34,851
Harland	1	22,108	30,027	22,669	24,711
Harland 23	1	36,056	46,149	34,371	37,812
Hickahala	1	26,403	33,247	39,944	36,554
Hickahala	2	52,377	35,031	60,740	58,953
Hotopha	1	111,480	133,137	71,568	20,867
Lick	1	42,837	61,262	52,443	25,842
Long	1	15,880	40,367	32,866	28,069
Long	2	39,410	33,859	25,513	11,017
Long	3	60,559	81,830	58,578	32,311
Long	4	36,171	18,180	16,014	24,612
Otoucalofa	1	26,409	22,171	26,278	32,545
Redbanks	1	53,303	42,414	51,215	49,923
Sykes	1	47,773	49,850	68,073	87,471
Worsham	1	79,347	90,796	83,030	79,569
Worsham	2	11,543	15,876	9,114	18,932
E. Worsham	1	52,761	55,610	38,390	101,079
M. Worsham	1	22,059	19,498	19,352	16,655
M. Worsham	2	25,061	18,343	3,575	16,186
M. Worsham	3	23,576	11,273	6,762	5,103
M. Worsham	4	8,033	9,680	7,715	8,477
W. Worsham	1	26,853	62,153	24,927	26,118
W. Worsham	2	44,312	43,218	33,152	27,539
W. Worsham	3	48,269	20,903	17,432	15,502
W. Worsham	4	140,714	58,983	66,282	72,063
ALL DATA					
Annual Sediment Yield (cubic yards)		1,111,192	1,104,061	923,684	894,080
Percentage of 1993 Sediment Yield			1%	17%	20%
REACHES WITHOUT GRADE CONTROL					
Annual Sediment Yield (cubic yards)		414,397	522,416	476,140	446,588
Percentage of 1993 Sediment Yield			-26%	-15%	-8%
REACHES WITH GRADE CONTROL					
Annual Sediment Yield (cubic yards)		696,795	581,645	447,544	447,492
Percentage of 1993 Sediment Yield			17%	36%	36%

6.2.2.2 Geotechnical Considerations

The above discussion focused only on the hydraulic aspects of design and siting of grade control structures. However, in some cases, the geotechnical stability of the reach may be an important or even the primary factor to consider when siting grade control structures. This is often the case where channel degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank height (Thorne and Osman, 1988). When this occurs, bank instability may be widespread throughout the system rather than restricted to the concave banks in bendways. Traditional bank stabilization measures may not be feasible in situations where system-wide bank instabilities exist. In these instances, grade control may be the more appropriate solution.

Grade control structures can enhance the bank stability of a channel in several ways. Bed control structures indirectly affect the bank stability by stabilizing the bed, thereby reducing the length of bankline that achieves an unstable height. With hydraulic control structures, two additional advantages are apparent with respect to bank stability: (1) bank heights are reduced due to sediment deposition, which increases the stability of the banks with regard to mass failure; and (2) by creating a backwater situation, velocities and scouring potential are reduced, which reduces or eliminates the severity and extent of basal cleanout of the failed bank material, thereby promoting self-healing of the banks.

An analysis of all cross sections surveyed each year during the period 1993 through 1996 with a bank angle greater than 50 degrees was made using the Darby and Thorne (1996a,b) method. Table 6.2 summarizes the results of the analyses.

Table 6.2 Bank Stability Analysis Summary

	1993	1994	1995	1996
Number of Cross Sections				
Total	28	25	26	41
With Grade Control	15	14	13	20
Without Grade Control	13	11	13	21
Failing Reach Length (m)				
With Grade Control	457	0	0	305
Without Grade Control	2,403	2,904	4,169	3,342
Total Volume Failing (cu. m)				
With Grade Control	1,463	0	0	1,527
Without Grade Control	6,488	10,456	15,424	22,059

These data show a significant increase in the number of cross sections greater than 50 degrees in 1996 over the previous three years, with an approximately equal number of the steeper banks occurring in reaches with and without grade control. The increase in steep banks may be due to several factors, however, no reason is adequately known to offer an opinion.

For reaches without grade control, the total volume of failure has increased by a factor of 3.4 for reaches without grade control as bank heights increase due to incision. By comparison, the total volume of failure for reaches with grade control has not significantly increased and has been zero for two of the four years. In 1996, the volume of steep bank cross section failure with grade control was only 7% of similar cross sections without grade control. The data appear to confirm the value of grade control in reducing bank height for bank stabilization in incising streams.

6.2.2.3 Flood Control Impacts

Channel improvements for flood control and channel stability often appear to be mutually exclusive objectives. For this reason, it is important to ensure that any increased post-project flood potential is identified. This is particularly important when hydraulic control structures are considered. In these instances the potential for causing overbank flooding may be the limiting factor with respect to the height and amount of constriction at the structure. Grade control structures are often designed to be hydraulically submerged at flows less than bankfull so that the frequency of overbank flooding is not affected. However, if the structure exerts control through a wider range of flows including overbank, then the frequency and duration of overbank flows may be impacted. When this occurs, the impacts must be quantified and appropriate provisions such as acquiring flowage easements or modifying structure plans should be implemented.

Another factor that must be considered when designing grade control structures is the safe return of overbank flows back into the channel. This is particularly a problem when the flows are out of bank upstream of the structure but still within bank downstream. The resulting head differential can cause damage to the structure as well as severe erosion of the channel banks depending upon where the flow re-enters the channel. Some means of controlling the overbank return flows must be incorporated into the structure design. One method is simply to design the structure to be submerged below the top bank elevation, thereby reducing the potential for a head differential to develop across the structure during overbank flows. If the structure exerts hydraulic control throughout a wider range of flows including overbank, then a more direct means of controlling the overbank return flows must be provided. One method is to ensure that all flows pass only through the structure. This may be accomplished by building an earthen dike or berm extending from the structure to the valley walls which prevents any overbank flows from passing around the structure (Forsythe, 1985). Another means of controlling overbank flows is to provide an auxiliary high flow structure which will pass the overbank flows to a specified downstream location where the flows can re-enter the channel without causing significant damage (Hite and Pickering, 1982).

6.2.2.4 Environmental Considerations

Projects must work in harmony with the natural system to meet the needs of the present, without compromising the ability of future generations to meet their needs. Engineers and geomorphologist are responding to this challenge by trying to develop new and innovative methods for incorporating environmental features into channel projects. The final siting of a grade control structure is often modified to minimize adverse environmental impacts to the system.

Grade control structures can provide direct environmental benefits to a stream. Cooper and Knight (1987) conducted a study of fisheries resources below natural scour holes and man-made pools below grade control structures in north Mississippi. They concluded that although there was greater species diversity in the natural pools, there was increased growth of game fish and a larger percentage of harvestable-size fish in the man-made pools. They also observed that the man-made pools provided greater stability of reproductive habitat. Shields *et al.* (1990) reported that the physical aquatic habitat diversity was higher in stabilized reaches of Twentymile Creek, Mississippi than in reaches without grade control structures. They attributed the higher diversity values to the scour holes and low-flow channels created by the grade control structures. The use of grade control structures as environmental features is not limited to the low-gradient sand bed streams of the southeastern United States. Jackson (1974) documented the use of gabion grade control structures to stabilize a high-gradient trout stream in New York. She observed that following construction of a series of bed sills, there was a significant increase in the density of trout. The increase in trout density was attributed to the accumulation of gravel between the sills which improved the spawning habitat for various species of trout.

Perhaps the most serious negative environmental impact of grade control structures is the obstruction to fish passage. In some cases, particularly when drop heights are small, fish are able to migrate upstream past a structure during high flows (Cooper and Knight, 1987). However, in situations where structures are impassable, and where the migration of fish is an important concern, openings, fish ladders, or other passageways must be incorporated into the design of the structure to address the fish movement problems (Nunnally and Shields, 1985). The various methods of accomplishing fish movement through structures are not discussed here. Interested readers are referred to Nunnally and Shields (1985), Clay (1961), and Smith (1985) for a more detailed discussion.

The environmental aspects of the project must be an integral component of the design process when siting grade control structures. A detailed study of all environmental features in the project area should be conducted early in the design process. This will allow these factors to be incorporated into the initial plan rather than having to make costly and often less environmentally effective last minute modifications to the final design. Unfortunately, there is very little published guidance concerning the incorporation of environmental features into the design of grade control structures. One source of useful information can be found in the following technical reports published by the Environmental Laboratory of the Corps of Engineers, WES (Shields and Palermo, 1982; Henderson and Shields, 1984; and Nunnally and Shields, 1985).

6.2.3 SUMMARY

The effectiveness of grade controls in stabilizing incised channels has been documented based on results from the DEC watersheds. Effectiveness in improving stream ecology has also been briefly documented through references. Additional benefits in reducing damage to infrastructure, improving flood control and other factors is discussed. All of these benefits require careful design, construction, and maintenance. However, the most important factor in enhanced grade control effectiveness may be the initial planning and goal identification for the incised channel rehabilitation project.

The above discussion illustrates that the design of grade control structures is not simply a hydraulic exercise. Rather, there are many other factors that must be included in the design process. For any specific situation, some or all of the factors discussed in this section may be critical elements in the final siting of grade control structures. It is recognized that this does not represent an all inclusive list since there may be other factors not discussed here that may be locally important. For example, in some cases, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive cookbook procedure for designing grade control structures that can be applied universally. However, consideration of each factor in an analytic and balanced fashion, and avoiding reliance on empircal procedures, can lead to effective and intelligent use of grade control structures.

6.3 FLOW CONTROL

Although bank stabilization and grade control are the primary structural methods utilized in channel rehabilitation, project goals often require the implementation of flow control. Unfortunately, predicting the channel response to flow control is extremely difficult. The complexity of channel response is illustrated in Lane's relationship (Figure 3.19), which indicates that a reduction in discharge, would allow a steeper slope to exist in the channel. Therefore, the degradational potential in a channel due to excessive slope would be minimized or even eliminated by the reduced discharge. However, this scenario is based on the assumption that the sediment load (Q_s) and the bed material size (D_{50}) remain unchanged. This is seldom the case since the trapping of sediment in the reservoir generally results in a reduced sediment supply downstream of the dam. According to Lane's relationship, a reduced sediment load downstream of a dam would result in a flatter slope that would result from bed degradation. Thus, the reduction in discharge and sediment load downstream of a dam tends to produce counteracting results. The ultimate channel response depends on the relative magnitude of the changes in discharge and sediment load, and on the downstream watershed characteristics such as tributary inputs, geologic controls, bed and bank materials, and existing channel morphology.

Because of these complexities, it is extremely difficult to predict the anticipated channel response to the construction of a dam. Therefore, it is difficult to develop definitive design criteria or guidance for the use of flow control as a means of accomplishing project goals. There are also numerous environmental consequences that must be considered. Generally, where flow control has been used for channel stabilization, it has been based primarily on past experiences with similar projects in the area. For example, studies by the Natural Resources Conservation Service in Mississippi showed that when over 60% of the Abiaca Creek watershed area in Mississippi was controlled by flood water retarding structures, the channel instability problems were significantly reduced (Water Engineering & Technology, 1989). Obviously, each watershed will behave differently, and the Abiaca results can not be applied regionally without further investigations. However, the results do provide some indication that channel stability can be improved by flow control.

Watson *et al.* (1997) found that the skewness of the flow distribution was strongly related to sediment yield. Skewness, as defined by Chow (1964) is a measure of the lack of symmetry of a distribution. For example, with the coefficient of skewness, C_s, at zero ($C_s = 0$), the distribution is symmetric; with $C_s > 0$, the distribution is skewed to the right with a long tail on the right side; and with C_s < 0 the distribution is skewed to the left. Figure 6.19 depicts the relationship developed by Watson *et al.* (1997) between the skewness of the flow distribution for the 15-minute data, and the annual sand yield per unit area for nine streams within the DEC watersheds in Mississippi. A trend of increasing sediment yield with increasing skewness is evident. The outlier, Hickahala Creek, has an anomalously high, and unexplained, sand yield. Figure 6.19 indicates that within the channelized DEC streams, the sand yield (tons per acre) increased by a factor of almost ten for a three-fold increase in skewness. As the peak discharge of a watershed increases due to land use change, channelization, or channel incision, the skewness increases. Conversely, the decrease in the peak discharges associated with flow control will reduce the skewness of the flow distribution, and result in a reduction in the downstream delivery of sediment.

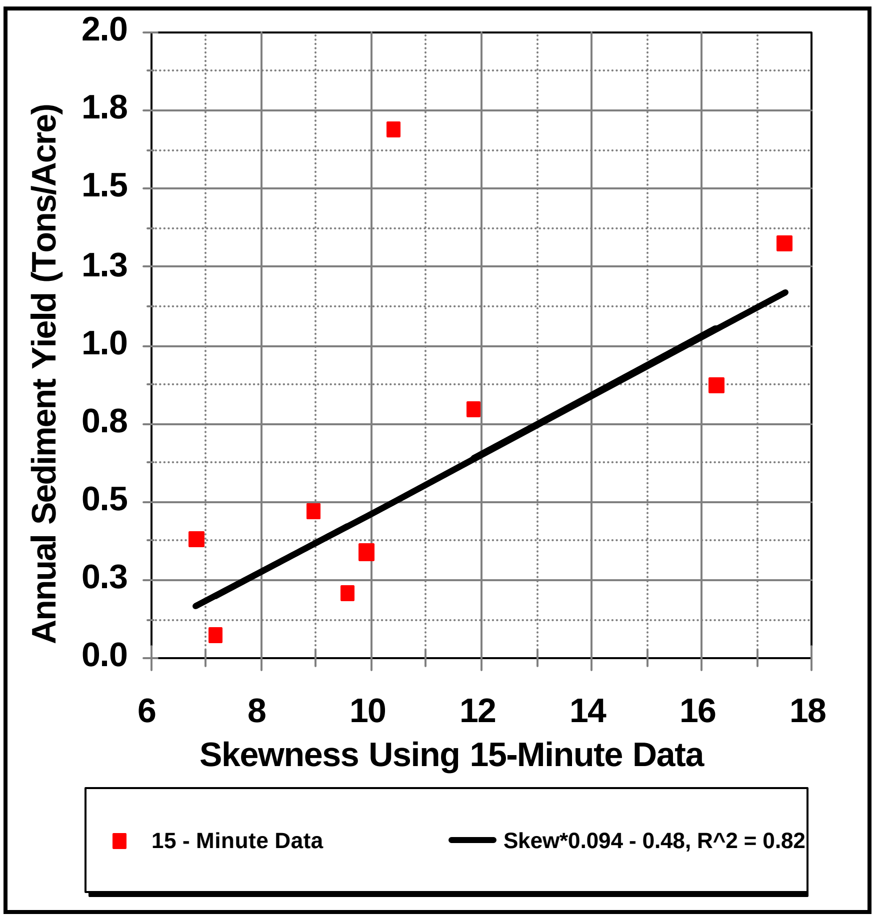


Figure 6.19 Annual Sediment Load as a Function of Flow Distribution Skewness

It also follows that for a given volume of a flood event, if the peak is of greater magnitude, the recession must be of shorter duration and of lower magnitude. At some low discharge, critical conditions exist for fish and other aquatic species. Many channelized streams do not have sufficient depth of flow for fish survival in summer baseflow. Karr *et al*. (1986) lists major flow regime factors that affect aquatic biota, which include: alteration of magnitude of high and low flows, increased maximum flow velocity, and a decrease in protected sites. These factors which are detrimental to aquatic biota are associated with increased skewness of the discharge. Therefore, reducing the skewness through flow control can offset some of the detrimental effects of channelization and channel incision.

In summary, increased storage in the watershed through flow control can be used to adjust hydrologic response to enhance aquatic and terrestrial habitat, improve channel stability, and to decrease sediment yield that can severely impair downstream flood control channels, reservoirs, and wetlands. However, the negative impacts associated with flow control such as decreased terrestrial habitat, reduced flushing flows for fish, decreased water temperatures, scour downstream of the dam, and blockage of fish passage must also be considered.

CHAPTER 7

CLOSING

Channel modification and channel improvement activities frequently have far reaching impacts upstream and downstream of the project site. Channel modification activities can adversely impact channel design features such as flood control, navigation, sediment control, and in-stream and riparian habitats. These activities are recognized by the Environmental Protection Agency (EPA) as a major source of nonpoint pollution. The Water Quality Act of 1987, section 101, includes the following policy statement: **It is the national policy that programs for the control of nonpoint sources of pollution be developed and implemented in an expeditious manner.**

Channel rehabilitation design should be applied not only to unstable systems that are causing problems, but also to new project designs where it is anticipated that changes made to the existing system will result in long-term stability problems. Because of the complexity of the channel rehabilitation design process, it is not easily summarized as a linear sequence of steps to be followed. The systematic approach taken by this manual is necessary for developing a workable project design that will function as intended. The design of these types of projects requires the synthesis and integration of extensive background data such as watershed geology, geography, sediment, hydrology, and hydraulics with analytic and empirical design procedures.

Unfortunately, at this time, no nationally recognized set of design and performance criteria exist to meet this mandate, nor is there a comprehensive manual that provides guidance for systematically approaching channel rehabilitation design. This manual has been developed as a reference to be used in channel rehabilitation training courses to be taught by the Waterways Experiment Station (WES) in cooperation with the EPA. The manual is designed to present an organized, systematic approach to channel rehabilitation, beginning with project planning and goal setting, through the preliminary project design phase. Specific alternative designs for channel rehabilitation are not presented in this manual. References for specific design information are provided in the text. The topics presented in this manual include:

1) The channel rehabilitation design process;
2) Geomorphic assessment and analysis of the proposed project area;
3) A summary of channel modification activities;
4) Fundamental engineering design computations to support systems analysis; and
5) Preliminary design procedures for stable channel design.

Closing

This manual is designed to be used by persons of a wide variety of experience and education. We would appreciate your comments and suggestions for enhancement of the document. Please contact:

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APPENDIX A

A PRACTICAL GUIDE TO EFFECTIVE DISCHARGE CALCULATION

A PRACTICAL GUIDE TO EFFECTIVE DISCHARGE CALCULATION

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ABSTRACT: This paper presents a procedure for calculating the effective discharge for rivers with alluvial channels. An alluvial river adjusts the bankfull shape and dimensions of its channel to the wide range of flows that mobilize the boundary sediments. It has been shown that time-averaged river morphology is adjusted to the flow that, over a prolonged period, transports most sediment. This is termed the effective discharge. While it may, under some circumstances, be possible to estimate the dominant discharge from the bankfull discharge, in practice, identification of bankfull stage in the field is often problematic. The dominant or channel-forming discharge may more reliably be found by calculating the effective discharge provided that the necessary data are available, or can be synthesized, and the calculations are properly performed. The procedure for effective discharge calculation presented here is designed to have general applicability, have the capability to be applied consistently, and to integrate the effects of physical processes responsible for determining the channel dimensions. An example of the calculations necessary and applications of the effective discharge concept are presented.

KEYWORDS

Bankfull Discharge, Effective Discharge, Hydraulic Geometry, Regime Theory, River Engineering, River Management, River Restoration, Stable Channel Design

INTRODUCTION: THEORIES AND CONCEPTS

While most engineers and river scientists recognize the conceptual limitations of dominant discharge theory, its application remains popular due to the versatility and utility of approaches that incorporate the principle that the cumulative effect of a range of discharges can be represented by a single flow. However, there is a great deal of uncertainty regarding both the terminology concerning dominant discharge and the best practice in its calculation. To alleviate both difficulties, this paper presents a definitive glossary of terms (Appendix II) and outlines the best practical procedure for calculating the effective or channel-forming discharge.

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Alluvial rivers have the potential to adjust their shape and dimensions to all flows that transport sediment, but Inglis (1941) suggested that, for rivers that are in regime, a single steady flow could be identified which would produce the same bankfull dimensions as the natural sequence of events. He referred to this flow as the dominant discharge. Based on field observations, Inglis (1947) took into consideration that the dominant discharge was approximated by flows at or about bankfull stage. This finding has been supported by subsequent research on the discharge controlling regime channel morphology (Nixon, 1959; Simons and Albertson, 1960; Kellerhals, 1967; Hey and Thorne, 1986) and studies of shallow overbank flows (Ackers, 1992; James and Brown, 1977).

To explain this phenomenon it is necessary to recognize that any local imbalance in the sediment budget must generate change in the morphology of a river through either erosion or deposition. Hence, to remain dynamically stable, the regime dimensions of the channel must be adjusted so that, over a period of years, sediment input and output balance. Over this time scale, Wolman and Miller (1960) showed that rivers adjust their bankfull capacity to the flow which, cumulatively, transports the most sediment and Andrews (1980) named that flow the effective discharge.

Wolman and Miller (1960) found that the effective discharge corresponds to an intermediate flood flow since very frequent minor floods transport too small a sediment load to have a marked impact on the gross features of the channel. While catastrophic events, which individually transport large sediment loads, occur too infrequently to be effective in forming the channel. The potential for large floods to disrupt the regime condition and cause major channel changes is recognized by this concept. However, large floods are not channel-forming provided that the return period of these extreme events is longer than the period required for subsequent, lesser events to restore the long-term, average condition (Wolman and Gerson, 1978).

Perennial rivers usually recover their long-term, average morphology within 10 to 20 years following a major event, principally because riparian and floodplain vegetation limits the impacts of major floods while vegetation regrowth encourages the processes of siltation involved in morphological recovery (Gupta and Fox, 1974; Hack and Goodlett, 1960). In semi-arid regions, the recovery period tends to be longer, reflecting the reduced effectiveness of sparse vegetation in increasing the channel's resilience to change and the sensitivity of the channel to the occurrence of relatively wet and dry periods (Schumm and Lichty, 1963; Burkham, 1972). In arid areas, the largest floods leave very long lasting imprints on the channel. Primarily because of the lack of vegetation and, secondly, because lesser events capable of restoring a regime condition rarely occur (Schick, 1974). For this reason, the dominant discharge concept is generally thought to be inapplicable to ephemeral streams in arid regions.

Equivalence between bankfull and effective discharges for natural alluvial channels that are stable has been demonstrated for a range of river types in different hydrological environments provided that the flow regime is adequately defined and the appropriate component of the sediment load is correctly identified (Andrews, 1980; Carling, 1988; Hey, 1997).

The equivalence of bankfull and effective discharges for stable channels suggests that either one could be used to define the channel-forming discharge. Also, in theory, this discharge could be determined

indirectly from an estimate of the return period for either the bankfull or effective discharge. In practice, problems often arise when attempting to use bankfull discharge to determine the dominant discharge. Problems center on the wide range of definitions of bankfull stage that exist (Williams, 1978). Although several criteria have been identified to assist in field identification of bankfull stage, ranging from vegetation boundaries to morphological breaks in bank profiles, considerable experience is required to apply these in practice, especially on rivers which have in the past undergone aggradation or degradation.

In many studies channel-forming discharge is estimated from the recurrence interval for bankfull discharge. Leopold and Wolman (1957) suggested that the bankfull flow has a recurrence interval of between one and two years. Dury (1973) concluded that the bankfull discharge is approximately 97% of the 1.58 year discharge, or the most probable annual flood. Hey (1975) showed that for three British gravel-bed rivers, the 1.5 year flow in an annual maximum series passed through the scatter of bankfull discharges measured along the course of the rivers. Richards (1982) suggested that, in a partial duration series, bankfull discharge equals the most probable annual flood, which has a one-year recurrence interval. For expediency, bankfull discharge is often assumed to have a recurrence interval of 1.5 years and Leopold (1994) stated that most investigations have concluded that the bankfull discharge recurrence intervals range from 1.0 to 2.5 years. However, there are many instances where the bankfull discharge does not fall within this range. For example, Pickup and Warner (1976) determined bankfull recurrence intervals may range from 4 to 10 years in the annual maximum series. Therefore, extreme caution must be used when estimating the dominant discharge using a flow of specific recurrence interval.

Although the channel forming discharge concept is not universally accepted, most river engineers and scientists agree that the concept has merit, at least for perennial and ephemeral streams in humid and semi-arid environments. There are three possible approaches to determining the channel-forming discharge: 1) bankfull discharge; 2) flow of a given recurrence interval, and; 3) effective discharge. Ideally, the method used should have general applicability, the capability to be applied consistently, and integrate the physical processes responsible for determining the channel dimensions. Of the three possible approaches listed above, only the effective discharge has the potential to meet these requirements. Another advantage of the effective discharge is that it can be calculated for post-project conditions, where the hydrologic regime may be significantly different from the existing regime due to the construction of dams, diversions, or there major channel improvement activities. Selection of the appropriate method will be based on data availability, physical characteristics of the site, level of study and time, and funding constraints. If possible, it is recommended that all three methods be used and cross-checked against each other to reduce the uncertainty in the final estimate.

HYDROLOGICAL DATA AND CALCULATIONS

Basic Principles

The range of flows experienced by the river during the period of record is divided into a number of classes and then the total amount of sediment transported by each class is calculated. This is achieved

by multiplying the frequency of occurrence of each flow class by the median sediment load for that flow class (Figure 1). Input data are:

i) flow data; and

ii) a sediment transport rating curve.

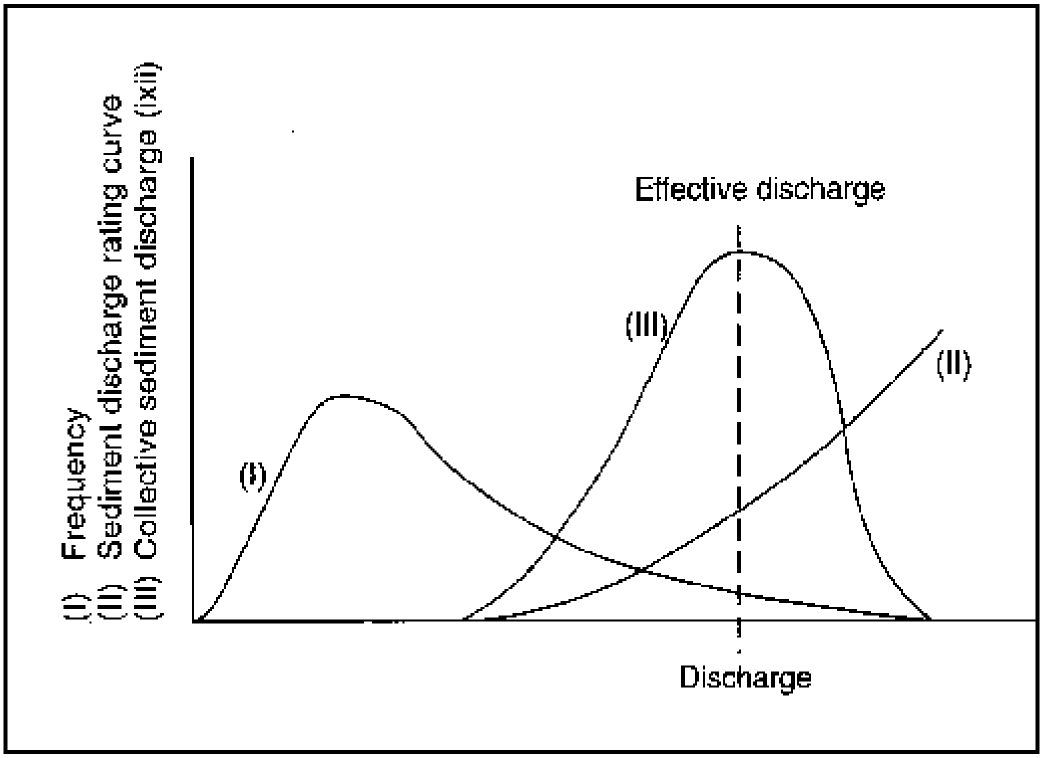

Figure 1 Derivation of Bed Material Load-discharge Histogram (iii) From Flow Frequency (i) and Bed Material Load Rating Curves (ii)

The calculated value of the effective discharge depends to some extent on the steps used to manipulate the input data to define the flow regime and sediment transport function. The procedure described here represents the best practice in this regard, based on extensive first-hand experience. Different procedures must be employed for gaged and ungaged sites.

Gaged Sites

At gaged sites, the first step is to group the discharge data into flow classes and determine the number of events occurring in each class during the period of record. This is accomplished by constructing a flow frequency histogram, which is a frequency distribution function of discharges measured at the gaging station. Three critical components must be considered when developing the flow frequency histogram: 1)

the number of discharge classes; 2) the time base for discharge averaging, and; 3) the length of the period of record.

Selection of Discharge Class Interval

The selection of class interval can influence the effective discharge calculation. Intuitively, it might be expected that the smaller the class interval and, therefore, the greater the number of classes, the more accurate would be the outcome. However, if too small an interval is used, discontinuities appear in the discharge frequency distribution. These, in turn, produce an irregular sediment load histogram with multiple peaks. Therefore, the selected class interval should be small enough to accurately represent the frequency distribution of flows, but large enough to produce a continuous distribution, with no classes having a frequency of zero.

There are no definite rules for selecting the most appropriate interval and number of classes, but Yevjevich (1972) stated that the class interval should not be larger than s/4, where s is an estimate of the standard deviation of the sample. For hydrological applications he suggested that the number of classes should be between 10 and 25, depending on the sample size. Hey (1997) found that 25 classes with equal, arithmetic intervals produced a relatively continuous flow frequency distribution and a smooth sediment load histogram with a well-defined peak. This indicates an effective discharge which corresponded exactly with bankfull flow. A smaller interval, and correspondingly larger number of classes, produced anomalous results. Particular care must be exercised on rivers where there is a high incidence of very low flows. In sand bed rivers, the low flows may be competent to transport the sediment. Under these circumstances, the effective discharge may be biased towards the lowest discharge class, and caution must be exercised, therefore, when using arithmetic class intervals.

Flows within the lowest discharge class are seldom normally distributed, being skewed towards the lower class boundary. The effect is that calculation of the sediment load transported by that class of flow, based on the median sediment transport rate, will overestimate the actual value. If this is a problem, it may be necessary to subdivide the lowest class interval into smaller classes.

An alternative approach for determining the class interval is to use the U.S. Geological Survey (USGS) flow duration procedure that divides the data into 35 classes. The lowest class is zero, with a class width of 0 to 0. The next class width is 0 to the minimum discharge value. The remaining 33 classes are determined by subtracting the logarithm of the minimum discharge from the logarithm of the maximum value, and dividing by 33, to define equal, logarithmic class intervals. The use of log-scale class intervals is attractive in that it divides the low discharges into more class intervals. This is useful because, on rivers where the flow duration curve is strongly skewed due to a high incidence of low flows, it generates approximately equal numbers of events in each. However, the discharge class intervals at the upper end of the distribution can be extremely large, artificially biasing the value of the effective discharge towards these flows. Caution must be exercised, therefore, when using logarithmic class intervals.

Time Base

Mean daily discharges are conventionally used to construct the flow duration curve. Although this is convenient, given the ready availability of mean daily discharge data from the USGS, it can, in some cases, introduce error into the calculations. This arises because mean daily values can under-represent the occurrence of short-duration, high magnitude flow events that occur within the averaging period, while over-representing effects of low flows. On large rivers the use of the mean daily values is acceptable because the difference between the mean and peak daily discharges is negligible. However, on smaller streams, flood events may last only a few hours, so that the peak daily discharge is much greater than the corresponding mean daily discharge. Excluding the flood peaks and the associated high sediment loads can result in underestimation of the effective discharge. Rivers with a high flashiness index, defined as the ratio of the instantaneous peak flow to the associated daily mean flow, are most likely to be affected. To avoid this problem it may be necessary to reduce the time base for discharge averaging from 24 hours (mean daily) to 1 hour, or even 15 minutes on flashy streams. When a shorter time base is used for the discharge data, it is necessary to use a corresponding time base for the sediment rating curve. For example, an investigation of discharge data for eleven USGS gaging stations in the Yazoo River Basin, Mississippi revealed that the annual yield of bed material calculated using mean daily discharge data was approximately 50% less than the yield calculated using 15-minute data (Watson *et al*., 1997). These are relatively small basins (drainage areas less than 1,000 km^2) with high rainfall intensities and runoff characteristics that have been severely affected by land-use change and channel incision. Consequently, hydrographs are characterized by steep rising and falling limbs, with events peaking and returning to base flow in much less than 24 hours.

In practice, mean daily discharge data may be all that are available for the majority of gaging stations and these data may be perfectly adequate. However, caution must be exercised when using mean daily data for watersheds with flashy runoff regimes and short-duration hydrographs. The use of 15-minute data to improve the temporal resolution of the calculations should be seriously considered whenever the available flow records allow it.

Period of Record

The period of record must be sufficiently long to include a wide range of morphologically-significant flows, but not so long that changes in the climate, land use or runoff characteristics of the watershed produce significant changes with time in the data. If the period of record is too short, there is a significant risk that the effective discharge will be inaccurate due to the occurrence of unrepresentative flow events. Conversely, if the period is too long, there is a risk that the flow and sediment regimes of the stream at the beginning of the record may be significantly different to current conditions. A reasonable minimum period of record for an effective discharge calculation is about 10 years, with 20 years of record providing more certainty that the range of morphologically significant flows is fully represented in the data. Records longer than 30 years should be examined carefully for evidence of temporal changes in flow and/or sediment regimes. If the period of record at a gaging station is inadequate, consideration should be given to developing an effective discharge based on regional estimates of the flow duration as outlined below.

Ungaged Sites

At ungaged sites, and gaged sites where records are found to be unrepresentative of the flow regime, it is necessary to synthesize a flow duration curve. There are two possible methods of doing this, using records from nearby gaging stations within the same catchment or hydrological region.

Catchment Flow Duration Curve Method (Basin-Area Method)

This method relies on the availability of gage data from a number of stations along the same river as the ungaged study site. First, flow duration curves for each gaging station are derived for the common period of record. Provided there is a regular downstream decrease in the discharge per unit watershed area, then a graph of discharge for a given exceedance duration against upstream basin area will produce a power function best fit regression line with negligible scatter. For example, Figure 2 shows this relationship for the River Wye, UK (Hey, 1975). The equations generated by this method enable the flow duration curve at an ungaged site on that river to be determined as a function of its upstream watershed area. Flow frequencies for selected discharge classes may then be extracted from the flow duration curve for the ungaged site.

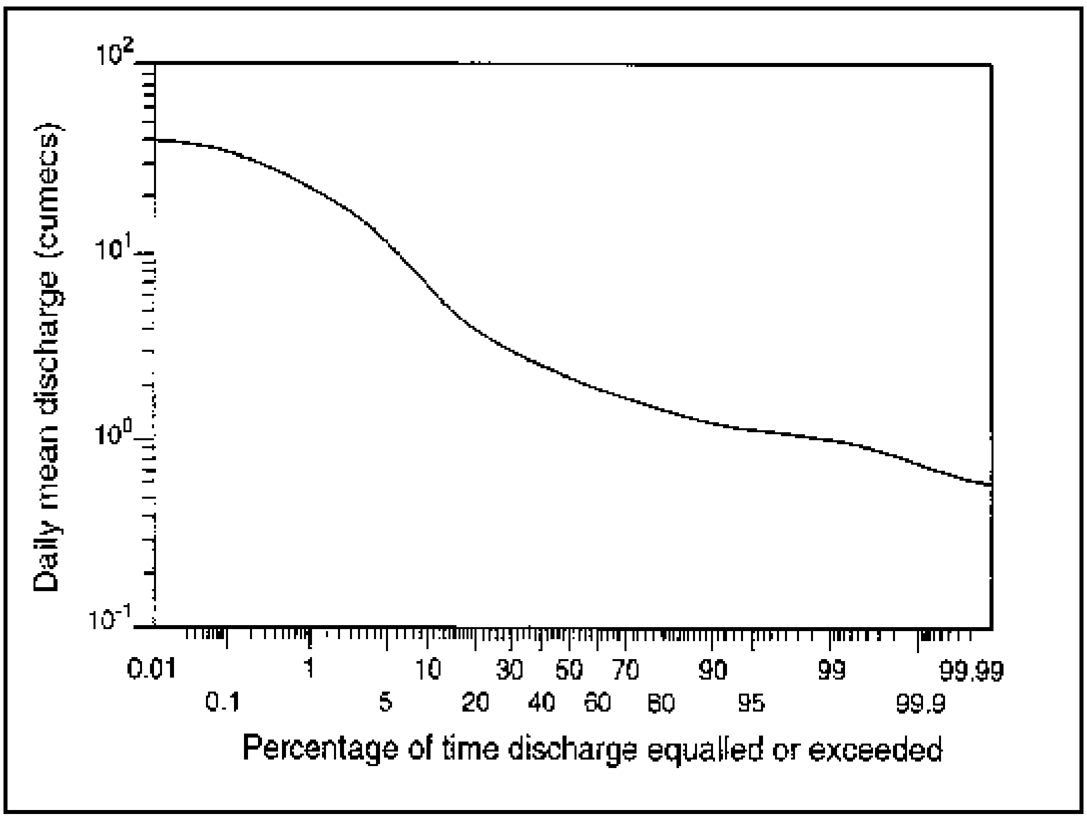

Figure 2 Downstream Daily Flow Duration Curves for the River Wye, UK Based on Data from Gaging Stations Collected Between 1937 and 1962 (adapted from Hey, 1975).

For ungaged sites on streams with only one gaging station, flow duration curves can be estimated for ungaged sites provided that the streams are tributaries to rivers where the relation between discharge and basin area conforms to a known power function. Estimates of the contributing flow to the mainstem can be obtained from the difference between discharges on the mainstem above and below the tributary junction. Discharge-basin area relations can then be derived for the tributary given the flow duration curve at the gaging station and the predicted curve at its confluence with the mainstem. However, this technique should not be used if there are distinct and abrupt downstream changes in the discharge per unit area for the watershed, due to tributaries draining different hydrological regions. In this case it would be preferable to use the regionalized duration curve method described next.

Regionalized Duration Curve Method

An alternative to the use of watershed area to generate a flow duration curve for an ungaged site is to use a regional-scaling method based on data from watersheds with similar characteristics. For example, Emmett (1975) and Leopold (1994) suggest using the ratio of discharge to bankfull discharge (Q/Q_b) as a non-dimensional index with which to transfer flow duration relationships between basins with similar characteristics. However, bankfull discharge does not necessarily have either a consistent duration or return period (see, for example, Williams, 1978). To get around this problem, a non-dimensional discharge index was proposed by Watson *et al.* (1997) using the regionalized 2-year discharge to normalize discharges (Q/Q_2).

For ungaged sites the 2-year discharge may be estimated from regionalized discharge frequency relationships developed by the USGS (1993) on the basis of regression relationships between the drainage area, channel slope, and slope length. These relationships are available for most American states. The dimensionless discharge index (Q/Q_2) can be used to transfer a flow duration relationship to an ungaged site from a nearby, gaged site. The gaged site may either be within the same basin, or an adjacent watershed. The steps involved in developing a regional flow duration relationship are:

i. Select several gaging stations and divide the discharge values of the flow duration relationship for each station by the respective Q_2 for that gage.

ii. Plot these ratios on a log-log graph. An example plot is shown in Figure 3, which is based on combined data for 10 gaging stations in watersheds in Mississippi used in the Demonstration Erosion Control (DEC) Project (Watson *et al.*, 1997). The regression analysis was performed assuming that discharges less than 1% of the Q_2 and with a probability of less than 1%, are morphologically insignificant and may be ignored.

iii. A flow duration curve for any ungaged site may then be computed by substituting the regionalized Q_2 for that site. Flow frequencies for selected discharge classes may then be extracted from the flow duration curve for the ungaged site.

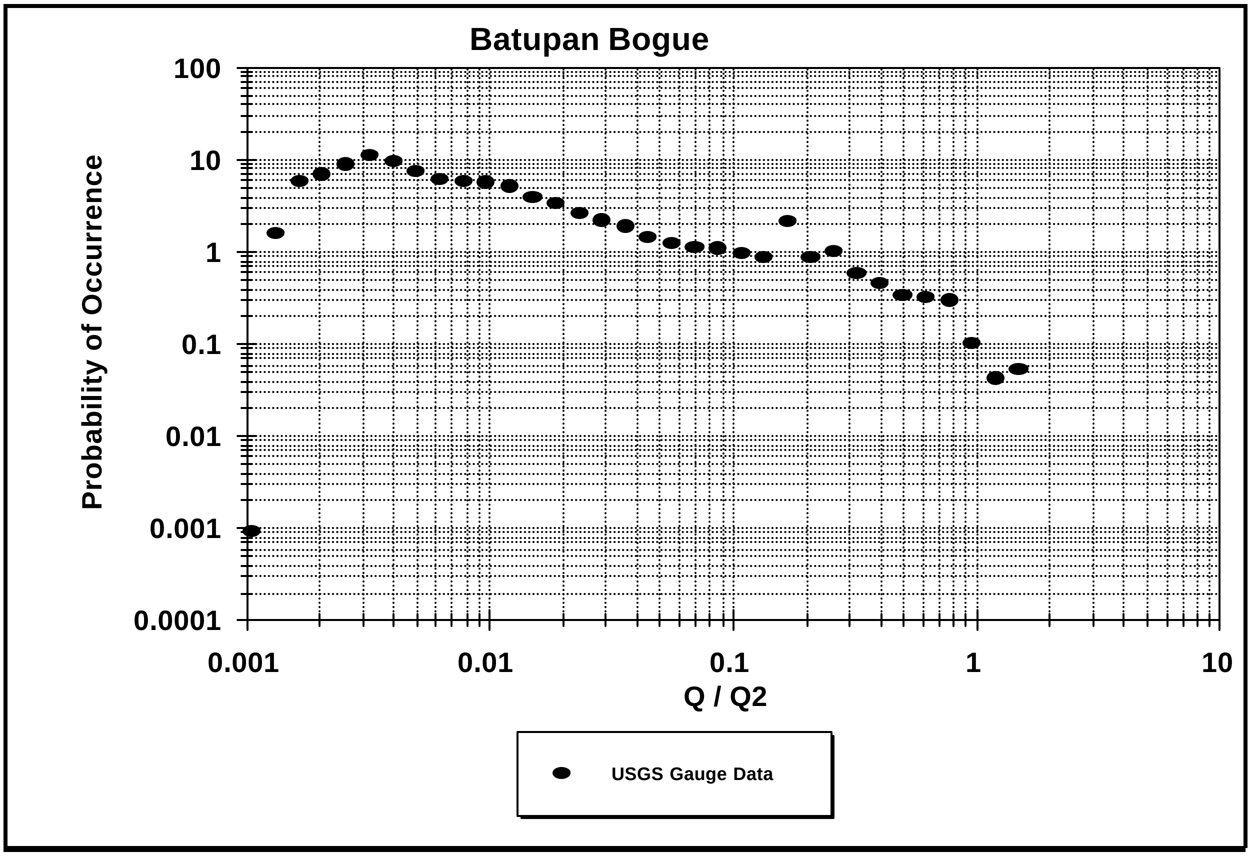

Figure 3 Regionalized Discharge Index for DEC Watersheds, Mississippi (adapted from Watson *et al.*, 1997)

Watson *et al.* (1997) tested the accuracy of these approaches. They found that the average error in bed material sediment yields at all the ungaged sites tested was only 2.8% when the method was used to transfer a flow duration relationship within a watershed, and 5.5% when it was used to develop a regional flow duration relationship.

SEDIMENT TRANSPORT DATA AND CALCULATIONS

Nature of the Sediment Load

The total sediment load of a stream can be broken down on the basis of measurement method, transport mechanism or source (Table 1). The transport of bed material load can be classified as bedload dominant, mixed load or suspended load dominant on the basis of the ratio of shear velocity to fall velocity (Julien, 1995). If this ratio is less than about 0.4, bedload is dominant. Between about 0.4 and 2.5, bed material loads move as a mixture of bedload and suspended load. Above about 2.5, suspension is the dominant transport mechanism. When discussing the sediment load of a stream, it is important to keep track of the terminology adopted and the nature of the load being discussed.

Table 1 Classification of the Total Sediment Load

Measurement Method (1)	Transport Mechanism (2)	Sediment Source (3)
Unmeasured Load	*Bed Load*	*Bed Material Load*
Measured Load	*Suspended Load*	*Wash Load*

Sediment Transport Data: Gaged Sites

In most alluvial streams the major features of channel morphology are principally formed in sediments derived from the bed material load. It is, therefore, the bed material load that should be used in an effective discharge calculation. At gaged sites the measured load usually represents the suspended load, but excludes the bed load. Under these circumstances, the coarse fraction of the measured load (generally the sand load - that is particles larger than 0.063 mm) should be used to derive a bed material load rating curve. If available, bed load data should be combined with the coarse fraction of the measured load to derive a bed material load rating curve.

Where a significant proportion of the bed material load moves as bed load (such as in gravel-bed rivers), but no measurements of bed load are available, it may be necessary to estimate the bed load. This may be achieved using a suitable bed load transport equation or the SAM hydraulic design package (Thomas *et al*., 1994). Similarly, at gaging stations with no measured sediment load data, a bed material sediment rating curve may be generated using appropriate sediment transport equations, or the SAM package.

Sediment Transport Data: Ungaged Sites

At ungaged sites it will be necessary to generate a bed material sediment rating curve. The application of a suitable sediment transport equation is vital and the SAM package is helpful because it includes guidance on the selection of equations best-suited to the type of river and bed material in question (Raphelt, 1990). For example, Watson *et al.* (1997) report an analysis of sediment discharge in Abiaca Creek in the Yazoo Basin using a HEC-2 computation of the hydraulic characteristics and the Brownlie (1981) sediment transport equation from SAM. Figure 4 shows the resulting sediment rating curve together with the measured sand fraction and a rating curve based on regression of the measured data. Close agreement is apparent between the Brownlie computation of the bed material load and the regression line based on the observed USGS sand fraction data.

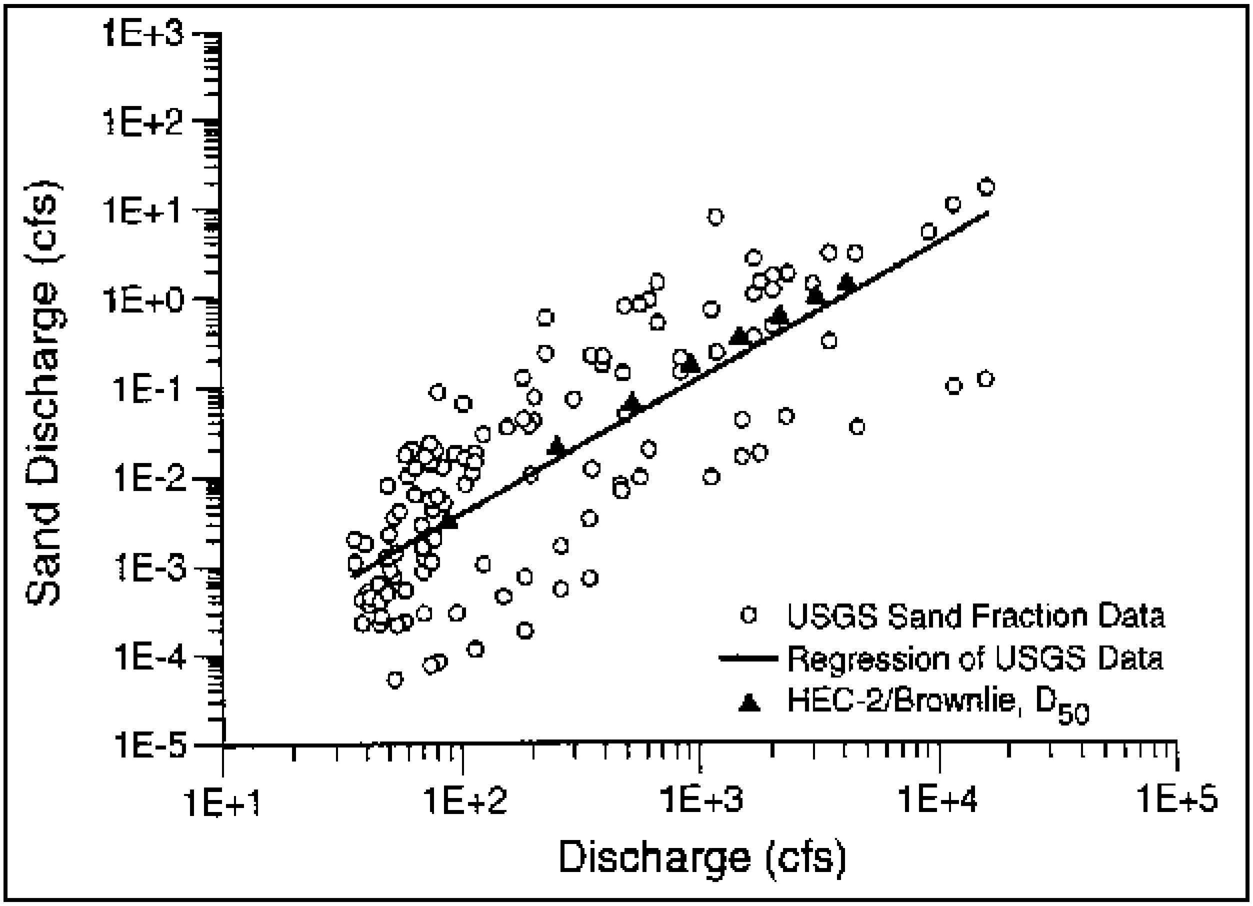

Figure 4 Comparison of Sediment Relationships for a Gage Site on Abiaca Creek, Mississippi (adapted from Watson *et al.,* 1997)

COMPUTATIONAL PROCEDURE

Overview

The recommended procedure to determine the effective discharge is executed in three steps. In Step 1, the flow frequency distribution is derived from available flow duration data (Figure 5). In Step 2, sediment data are used to construct a bed material load rating curve (Figure 6). In Step 3, the flow frequency distribution and bed material load rating curve are combined to produce a sediment load histogram (Figure 7), which displays sediment load as a function of discharge for the period of record. The histogram peak indicates the effective discharge.

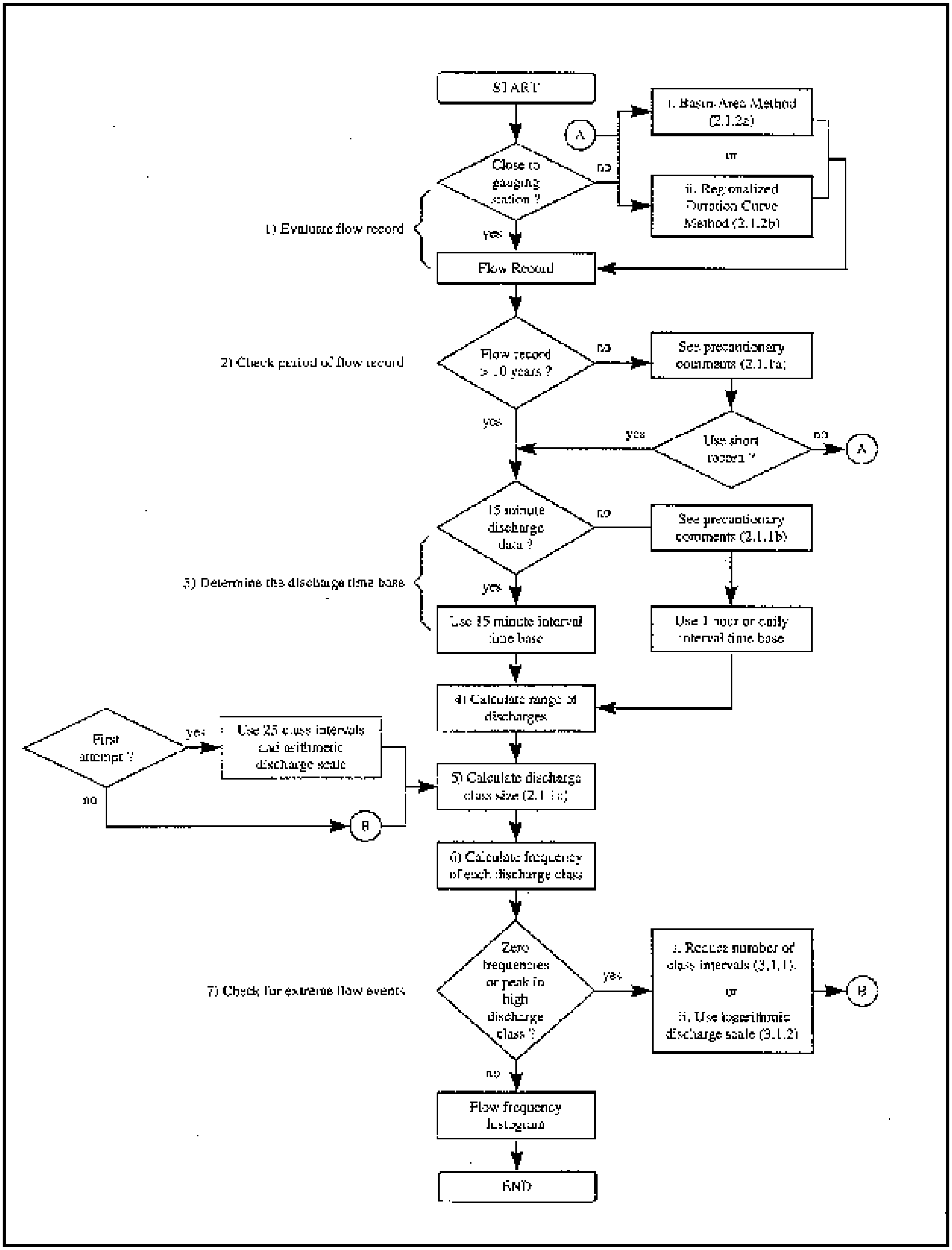

Figure 5 Procedure for Generating a Flow Frequency Histogram

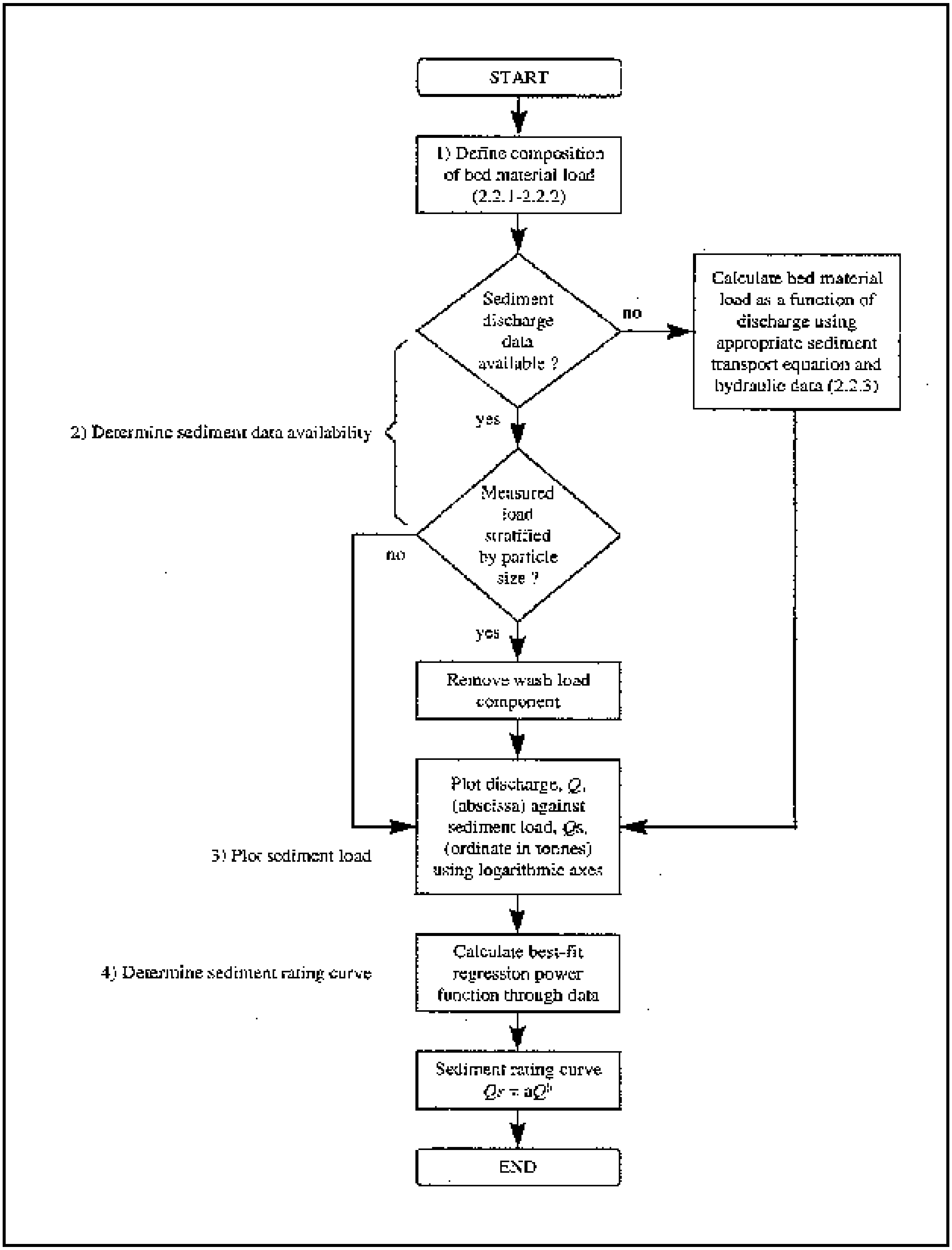

Figure 6 Procedure for Generating a Bed Material Load Rating Curve

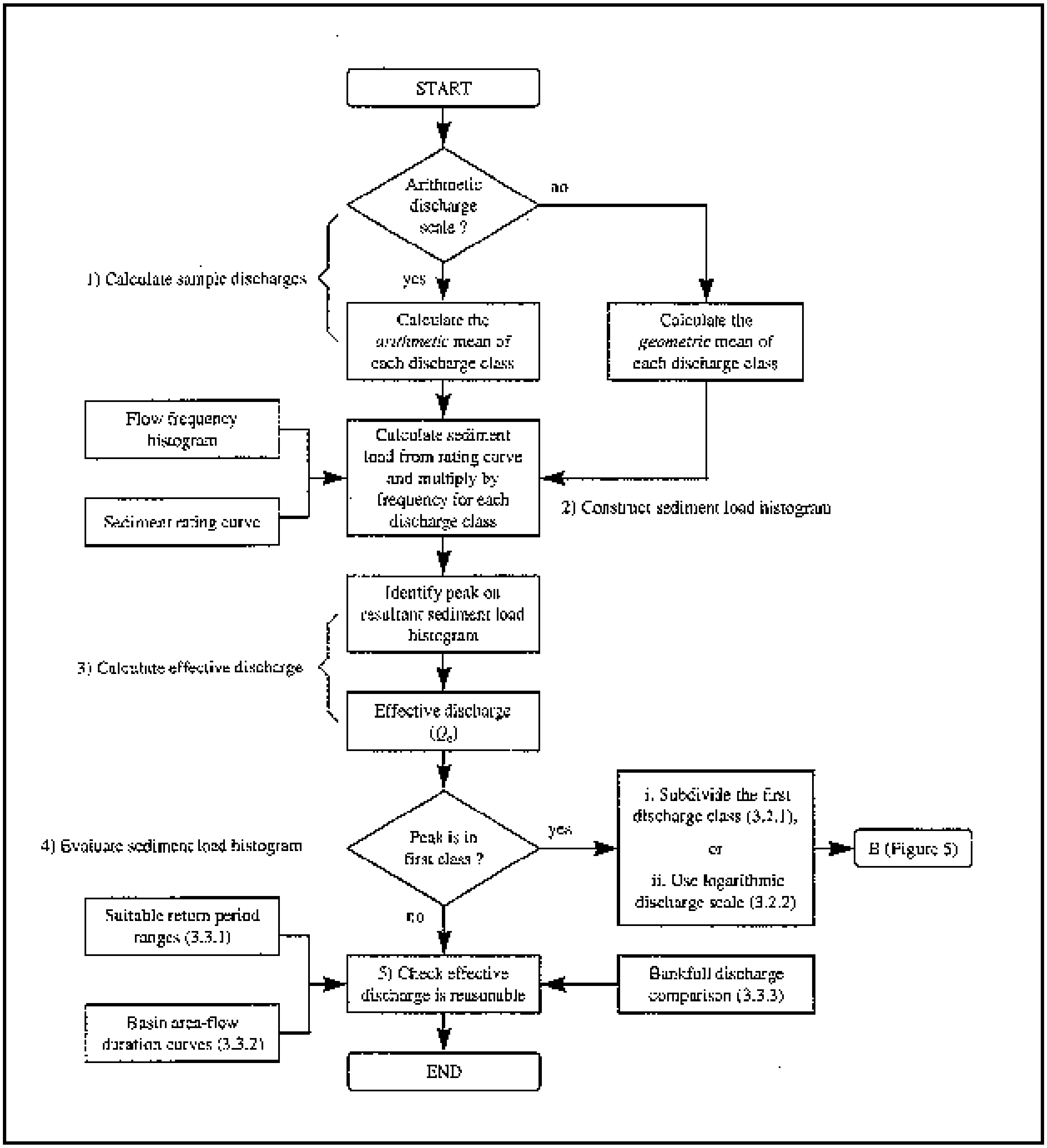

Figure 7 Procedure for Generating a Bed Material Load Histogram

Step 1 - Flow Frequency Distribution

1) Evaluate Flow Record

The flow record is a historic record of discharges at a gaging station. The record from a single gaging station can be used to develop the flow frequency distribution if the gage is in close proximity to the study site and the discharge record at the gage is representative of the flow regime there. If a gaging record is either unavailable or unrepresentative, the flow frequency distribution can be derived using either the basin-area or regionalized duration curve method.

2) Check the Period of Record

It is recommended that the length of period of record be at least 10 years and that measurements be continuous to the present day. Discharge data can still be used if there are short gaps in the record, but caution must be exercised when collecting data from a discontinuous record. The flow frequency curve will not be representative of the natural sequence of flows over the medium term if the length of record is less than 10 years or if the record has been influenced by changes in the watershed runoff regime. If this is the case, a flow duration curve should be developed using either the basin-area or regionalized duration curve method.

3) Determine the Discharge-Averaging Time Base

To construct the flow frequency distribution, the time base should be sufficiently short to ensure that short-duration, high magnitude events are properly represented. If 15-minute data are unavailable, then either 1-hour or mean daily data can be used, but caution must be exercised when using mean daily data to develop a flow frequency distribution for a stream which exhibits a flashy regime.

4) Calculate Discharge Range

The range of discharges is calculated by subtracting the minimum discharge in the flow record from the maximum discharge.

5) Calculate Discharge Class Interval

It is recommended that the initial attempt to construct the flow frequency distribution should use 25, arithmetic class intervals. Therefore, the class interval is the discharge range, calculated in Step (4), divided by 25. The class interval should not be approximated by rounding. The relative proportions of the bed material load moving in suspension and as bed load should be estimated during site reconnaissance. For rivers in which the bed material load moves predominantly as suspended load, the first discharge class goes from zero to the class interval, the second class is determined by adding the class interval to the upper value of the previous class, and so on until the upper limit of the discharge range is reached. For gravel-bed rivers, where bed material load moves predominantly as bed load, the minimum discharge used in

generating the flow frequency distribution should be set equal to the critical discharge for the initiation of bed load transport.

6) <u>Calculate Flow Frequency Distribution</u>

The frequency of occurrence for each discharge class is determined from the record of observed flows. The frequency units should reflect the time base in the flow record. For example, if mean daily flows have been used then the frequency is expressed in days. If a regional flow duration curve has been developed for an ungaged site, then the frequency for each discharge class must be calculated using the equation for the curve, which is usually a power function. This can be achieved by calculating the geometric mean discharge of each discharge class and deriving the frequency from the equation of the curve.

7) <u>Check For Extreme Flow Events</u>

It is recommended that all discharge classes display flow frequencies greater than zero and that there are no isolated peaks in individual classes at the high end of the range of observed discharges. If this is not the case, it is likely that either the class interval is too small for the discharge range, or the period of record is too short. Both zero frequencies and extreme flow events (outliers), can be eradicated by reducing the number of classes or using logarithmic class intervals, as described in the later section on Evaluation and Troubleshooting, but noting the cautions in each case. In either case, Steps 5 and 6 are repeated to generate the flow frequency distribution for the new class intervals.

Step 2 - Bed Material Load Rating Curve

1) <u>Determine Sediment Data Availability</u>

Sediment transport data are required to generate the bed material load rating curve. These data may be obtained from measurements at a gaging station if the gage is in close proximity to the study site and the sediment record at the gage is representative of the sediment load there. Otherwise, sediment transport data must be derived for the study site.

2) <u>Define Composition of Bed Material Load</u>

It is recommended that wash load (generally defined as particles less than 0.063 mm) be excluded from the data set used to develop the sediment rating curve. If the bed material load moves both as bed load and suspended load, then bed load and suspended load measurements are required to determine the bed material load. If measured load data are insufficient, appropriate equations in a hydraulic design package, for example SAM (Thomas *et al*., 1994), can be used to generate bed material loads for selected discharges.

3) Plot Bed Material Load Data

The bed material load (y-axis) is plotted as a function of discharge (x-axis) in a scattergraph, with both axes on logarithmic scales, carefully matching the units of sediment load to the time base for the discharge frequency distribution.

4) Determine Sediment Rating Curve

A power function best-fit regression line should be fitted to the data in the scatter graph to produce a bed material load function of the form:

$$Q_s = aQ^b \tag{1}$$

where, Q_s = bed material load discharge, Q = water discharge, a = regression constant, and b = regression coefficient.

Step 3 - Bed Material Load Histogram

1) Calculate Representative Discharges

The discharges used to generate the bed material load histogram are the mean discharges in each class of the flow frequency distribution. If discharge class intervals are arithmetic, the representative discharges are the arithmetic means of each class. If the discharge class intervals are logarithmic, the representative discharges are the geometric means of each class.

2) Construct the Bed Material Load Histogram

The bed material transport rate for each discharge class is found from the rating curve equation. This load is multiplied by the frequency of occurrence of that discharge class to find the total amount of bed material transported by that discharge class during the period of record. Care should be taken to ensure that the time units in the bed material load rating equation are consistent with the frequency units for the distribution of flows. The results are plotted as a histogram.

3) Find the Effective Discharge

The bed material load histogram should display a continuous distribution with a single mode (peak). If this is the case, the effective discharge corresponds to the mean discharge for the modal class (that is the peak of the histogram). If the modal class cannot be readily identified, the effective discharge can be estimated by drawing a smooth curve through the tops of the histogram bars and interpolating the effective discharge from the peak of the curve.

EVALUATION AND TROUBLESHOOTING

Derivation of an effective discharge is not a routine exercise and it is vital that at the end of the procedure, the output is evaluated to ensure that the calculated effective discharge is a reasonable value for the project river at the study site. This section of the paper presents a series of evaluations that should always be performed as part of quality assurance in deriving an effective discharge, together with guidance on troubleshooting some of the more common problems.

Problems with Lowest Discharge Class

When a significant proportion of the recorded discharges fall within the first arithmetic class interval of the flow frequency distribution, the range of discharges is inadequately represented and it is likely that the computed effective discharge will be significantly underestimated. In practice, this is likely to be the case for streams that display a highly skewed distribution of flow events, for example, rivers in semi-arid environments, channelized streams, or incised channels. Under these circumstances, it is advisable to modify the flow frequency distribution to better represent the range of low discharges, using one of the two approaches outlined below.

Use Logarithmic-Scale Class Intervals

The first solution is to replace the arithmetic class intervals with logarithmic ones. For example, Watson *et al*. (1997) demonstrated that this was the preferred solution for Hotopha Creek in the Yazoo Basin, Mississippi. In this example, approximately 97% of the recorded discharges fell within the lowest, arithmetic class interval. This resulted in a computed effective discharge of 8 m^3/s, which was known from experience to be too low a flow to have any morphological significance. When logarithmic class intervals were adopted, the smaller flow events were distributed between several classes and the effective discharge was found to be 234 m^3s^{-1} which was identified as a morphologically significant flow.

Use of logarithmic class intervals solves problems at the low end of the discharge range, but care should be exercised because large flow events are grouped into a few, very large class intervals, which may result in inadequate precision if the effective discharge is found to be at the high end of the range.

Sub-divide the Lowest Class

An alternative solution is to subdivide the lowest discharge class into a number of equal sub-classes to ensure that the discharge events within each sub-class are more normally distributed. This will also address any problems relating to bed load transport, since flows below the threshold discharge for transport will be discounted. After completing this exercise, the total bed material load transported within the lowest discharge class should be found by summing the sediment loads for the subdivisions, to facilitate plotting of the bed material load histogram.

The effectiveness of this correction can be illustrated by reference to effective discharge calculations for stations at Marmarth and at Medora on the Little Missouri River reported by Hey (1997). Initially, the effective discharge was in the lowest class interval at both sites with values of 16.5 and 22.8 m^3s^{-1}, respectively. However, following sub-division of the lowest class, the effective discharge at each site corresponded to bankfull flow, with values of 68 and 90 m^3s^{-1}, respectively.

Problems with Outliers

Discharge records, especially those based on mean daily values, can contain distinct gaps in the higher discharge categories due to the averaging process. This is particularly likely on rivers with flashy hydrographs or if there has been an extremely large flood event during a relatively short period of record. The use of arithmetic discharge class intervals can produce a discontinuous flow frequency distribution that, in turn, generates an irregular bed material load histogram with outliers that reflect the transport associated with individual flood events. The danger is that the wrong peak may be selected to represent the modal class, leading to serious overestimation of the effective discharge. Under these circumstances, it is advisable to modify the flow frequency distribution using one of the two approaches outlined below.

Reduce the Number of Class Intervals

Outliers are readily apparent when using arithmetic class intervals through the presence of gaps in the bed material load histogram and excessively proportions of the load being transported in an isolated, high discharge class. They can be removed by increasing the class interval and reducing the number of classes to reduce the number of discharge classes with zero bed material load and smooth the histogram. For example, Hey (1997) reported that use of 40 class intervals for the White River upstream of Mud Mountain Dam, resulted in an effective discharge that was unrealistically high (387 m^3s^{-1}). Reducing the number of classes to 25, resulted in an effective discharge of 81 m^3s^{-1}, which corresponded to bankfull flow (Hey, 1997).

Use Logarithmic Class Intervals

Reducing the number of classes while maintaining an acceptable arithmetic class interval does not always eliminate all zero flow frequencies. In this case, the option of using logarithmic class intervals should be considered, as this will almost certainly solve the problem.

Checking the Effective Discharge

Guidance on Return Periods for the Effective Discharge

Return periods for effective discharges are expected to vary between study sites depending on the flow and sediment transport regime of the individual river or reach. For sites where annual maximum series flood flow data are available, the return period of the calculated effective discharge may be checked to ensure that it lies within acceptable bounds. Unfortunately, there is very limited information available

regarding the return period of the effective discharge for stable rivers. Experience indicates that it lies usually within the range of 1.01 and 3 years, with a preponderance between 1.01 - 1.2 years, irrespective of the type of river (Hey, 1994, 1997). On this basis, effective discharges with return periods outside the range of approximately 1 to 3 years should be queried.

Basin Area - Flow Duration Curve

The percentage of the time that the effective discharge is equalled or exceeded should be compared to the expected range of values reported in the literature. For example, Figure 8 presents a log-log plot of the flow duration of effective discharge as a function of drainage area for several U.S. rivers (Andrews, 1980, 1984; Biedenharn *et al*., 1987). The graph can be used to assess whether the duration of the effective discharge computed using the method described in this paper is consistent with the results of other studies. It is not intended that this graph be used to predict effective discharge as a function of drainage area, as large errors are likely to result from this application.

Check Effective Discharge against Bankfull Discharge

The effective discharge should be compared to the bankfull discharge. This can be accomplished by identifying the bankfull stage during stream reconnaissance and calculating the corresponding discharge using an available stage-discharge curve or the slope-area method. For naturally stable channels that are in regime, the effective and bankfull discharges should coincide. If the effective discharge is substantially different to the bankfull discharge, the results should be queried.

EXAMPLE: EFFECTIVE DISCHARGE CALCULATION FOR THE MISSISSIPPI RIVER AT VICKSBURG

Flow Frequency Distribution

Discharge data were obtained from the Vicksburg gage for the period 1950 to 1982. This period of record was selected as it encompasses the period when sediment loads were routinely measured at the gaging station. The record contains a wide distribution of flows including both low and high runoff years and with discharges ranging from about 4,200 to just over 56,600 m^3s^{-1}. On this very large river, mean daily discharges do not differ significantly from instantaneous discharges and so the use of mean daily values was acceptable in the production of the flow frequency distribution (Figure 9).

Bed Material Load Rating Curve

Sediment transport data were also obtained from the Vicksburg gaging station. The period of record is 1969 to 1979, as this was the only period for which measured sediment transport records were available. On average, sediment load was measured weekly. Robbins (1977) provides a detailed

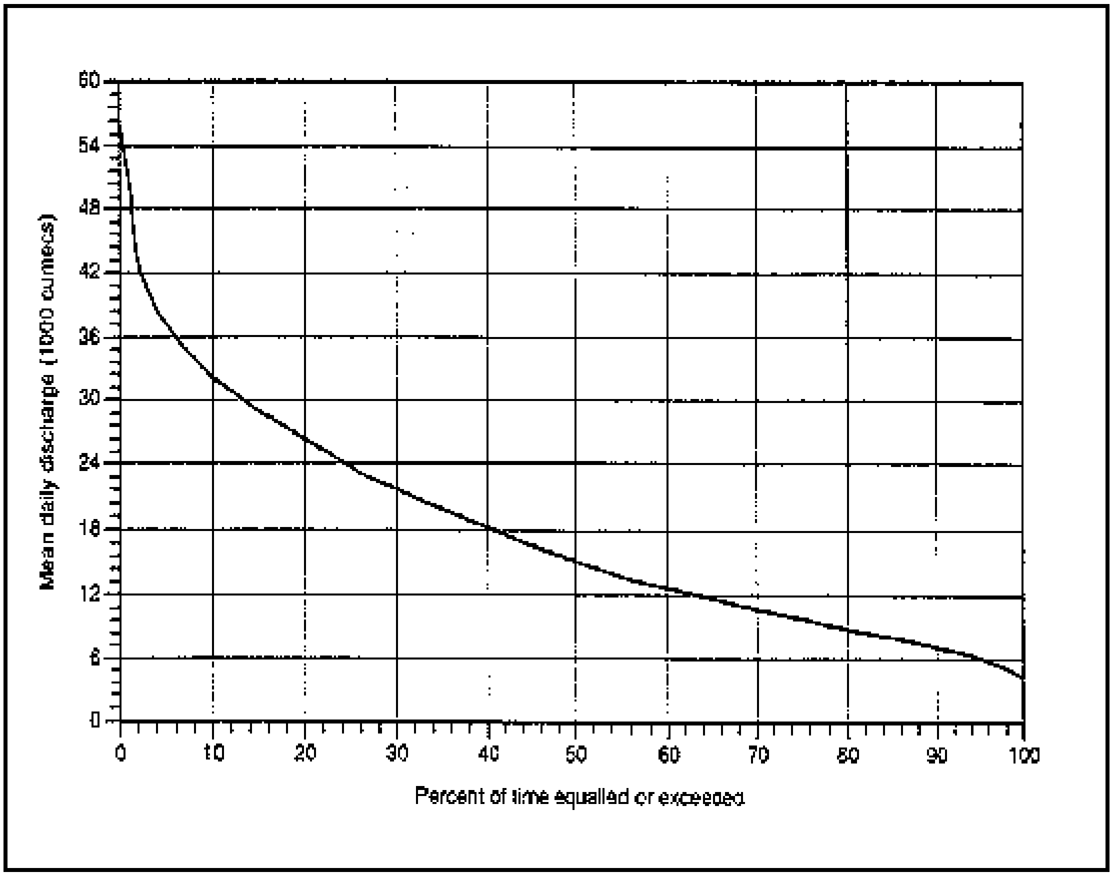

Figure 9 Flow Duration Curve for Mean Daily Discharge: Lower Mississippi River at Vicksburg, 1950-1982 (adapted from Biedenharn and Thorne, 1994)

description of the sediment measurement program on the lower Mississippi. The period of sediment records includes both low runoff years and several events of high magnitude and long duration, so that the full range of sediment transporting flows is represented in the measured data.

The measured sediment loads were divided into two components: 1) silt load consisting of particles less than 0.062 mm, and; 2) sand load consisting of particles coarser than 0.062 mm. The bed of the lower Mississippi River is formed in sand and so the sand fraction of the measured load was taken to represent the bed material load. The silt load was taken to represent "wash load" for the lower Mississippi and was excluded from the analysis.

There are no measurements of bed load for the lower Mississippi River, but according to the calculations of Toffaleti (1968) the bed load comprises less than 5% of the total sand load. Hence, it was deemed to be acceptable to ignore the bed load and to take the measured sand load as indicative of the bed material load. The measured sand load data were used to construct a sand load rating curve for the study site (Figure 10). Regression analysis of sand load as a function of discharge produced a coefficient of determination, r^2, of 0.82 and defined the bed material load rating curve as:

$$Q_s = 0.00000513\ Q^{2.42} \tag{2}$$

where: Q_s = sand load (tons day^{-1}), and Q = mean daily discharge (m^{3}s^{-1}).

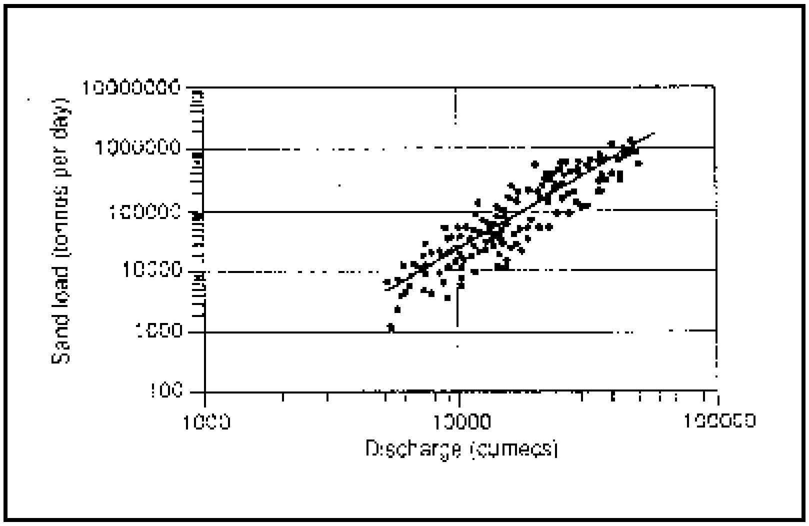

Figure 10 Sand Load Rating Curve: Lower Mississippi River at Vicksburg for 1969-1979 (adapted from Biedenharn and Thorne, 1994).

Bed Material Load Histogram

The data in the flow duration curve were divided into 50 equal classes ranging from 5 to 55,000 m^{3}s^{-1} and with a class width of 1,000 m^{3}s^{-1}. The bed material transport rate for each discharge class (Q_s) was found from Eq. (2), with Q equal to the arithmetic mean discharge for that class. The quantity of bed material load (in tons) transported by each discharge class was calculated by multiplying the frequency of each class (in days) by the bed material transport rate for the average discharge (in tons day^{-1}). The resultant histogram is plotted in Figure 11.

Effective Discharge Determination

The peak of the histogram in Figure 11 is defined by the mean discharge of the modal class, which is 30,000 m^{3}s^{-1}. This defines the effective discharge.

Check if Effective Discharge is Reasonable

In Biedenharn and Thorne's (1994) study, the effective discharge calculation was also performed for gaging stations at Arkansas City (upstream of Vicksburg) and Natchez (downstream of Vicksburg). No major tributaries enter the Mississippi between these stations. Hence, it would be expected that the

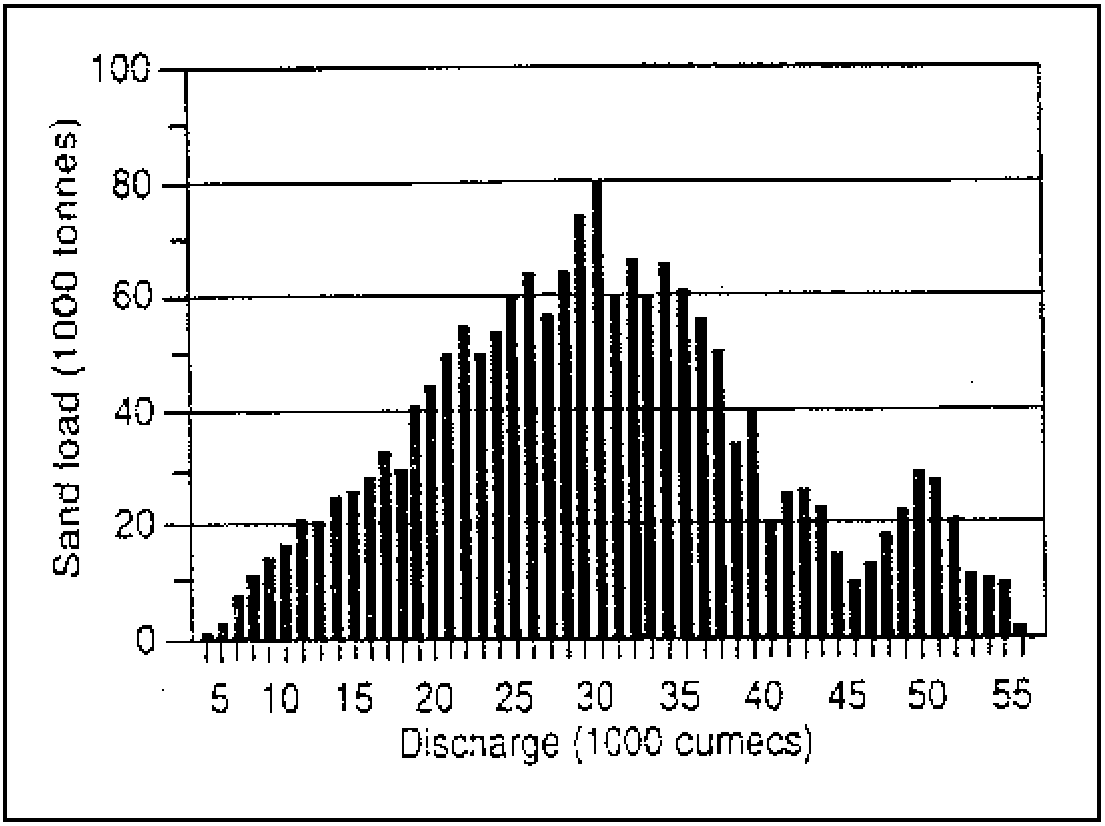

Figure 11 Bed Material Load Histogram: Lower Mississippi River at Vicksburg (adapted from Biedenharn and Thorne, 1994).

effective discharge should be the same at all three sites. This was in fact the case, illustrating consistency in the effective discharge analysis using three separate flow duration and sediment transport records.

Comparison of the water surface profile at the effective discharge (30,000 m^3s^{-1}) to the long-channel distribution of bank top elevations is illustrated in Figure 12. The graph shows that bank top elevations are highly variable and can differ by 3 m or more between adjacent cross-sections. This makes it difficult to assign a value to bankfull discharge for the reach. However, comparison of the water surface profile for the effective discharge to the bank top data indicates that the effective discharge forms a very good lower bound to the scatter, indicating that the capacity of the channel is adjusted just to contain flows up to and including the effective flow. As discharge increases beyond the effective flow, water begins to spill over the bank tops at more and more locations. The return period for the effective discharge (equal to or just less than one year) is consistent with the usual range of 1 to 3 years, and its flow duration (equalled or exceeded 13% of the time) is as expected for a river with a drainage area of approximately 3 million km^2 (see Figure 8). These checks indicate that the calculated effective discharge is very reasonable and they support the accuracy of the analysis, including the necessary assumptions concerning the use of measured sand load to represent the bed material load.

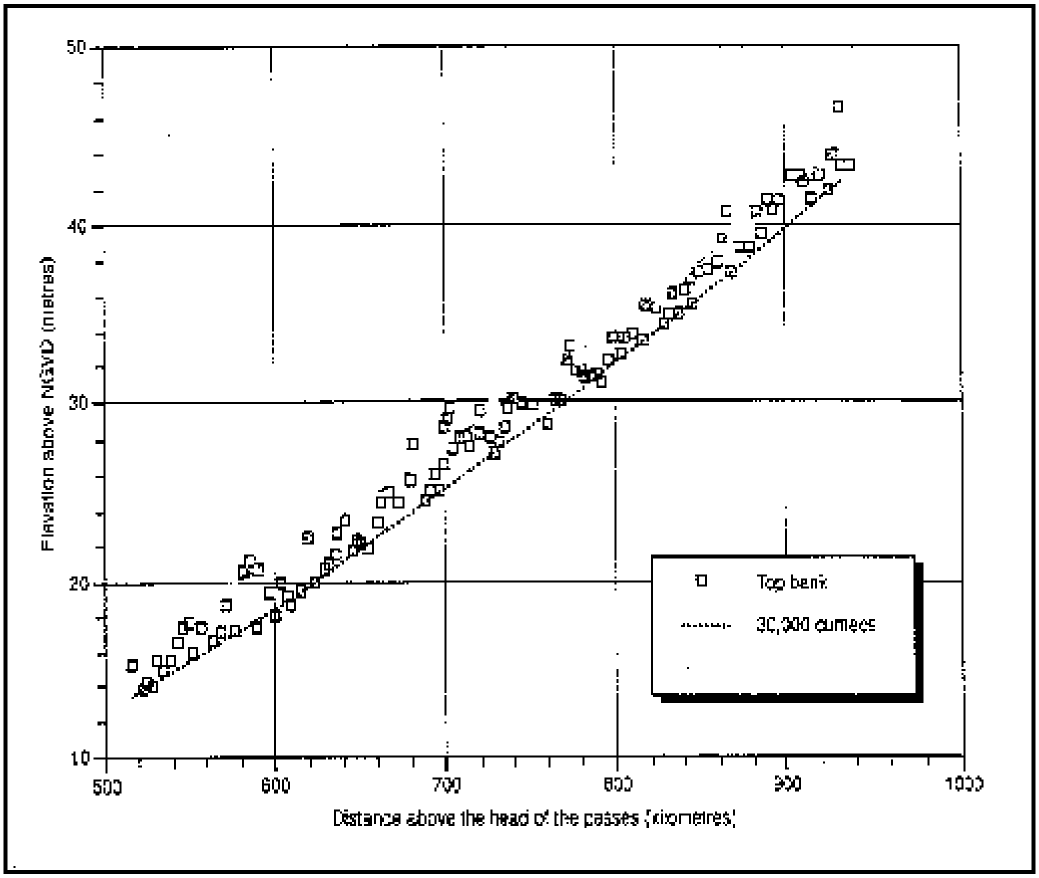

Figure 12 Long-channel Variation in Bank Top Elevations: Lower Mississippi in Study Reach (adapted from Biedenharn and Thorne, 1994).

APPLICATIONS

Application 1: Engineering-geomorphic Study of the River Blackwater, UK

The need for morphological studies to support sustainable engineering and management of rivers is now generally accepted (Gardiner, 1991; Downs and Thorne, 1996). Calculation of the effective discharge and application of the principles of hydraulic geometry analysis can be useful in developing a sound understanding of the morphological status and stability of an alluvial stream, as a component in an engineering-geomorphic study. The utility of this approach can be illustrated using a case study of the River Blackwater, UK. The study is reported in detail in a report by Wallingford (1992) and summarized in a paper by Thorne *et al.* (1996).

The River Blackwater is a lowland stream in southeast England. In the 1960s and 1970s the channel was modified by engineering works installed as part of a series of flood defence and land drainage schemes. Subsequently, the channel required a heavy maintenance regime to maintain a sufficient capacity

for flood flows. More recently, steps have been taken under the Blackwater Catchment Plan to improve stream habitats by softening the impacts of engineering works, modifying the maintenance regime, and enhancing the channel environment.

Under plans to restore the environmental function of the river, a geomorphological study was performed to establish how the existing, engineered channel differed morphologically from a natural, regime channel. An effective discharge calculation was performed using data from a nearby gaging station and the "expected" morphology for a natural, regime channel was established by applying the hydraulic geometry equations of Hey and Thorne (1986). Stream reconnaissance was performed to establish the morphology and bankfull dimensions of the existing, engineered channel using the method reported by Thorne (1998).

The effective discharge calculation showed that the dominant discharge was 3.65 m^3s^{-1}, compared to a bankfull capacity observed in the current channel of 16.4 m^3s^{-1}. The main morphological parameters calculated and observed are listed in Table 2. Contrasts between the regime and engineered channel parameters were used to support the conclusion that the engineered channel was over-large in width, depth, and cross-sectional area, and that in-channel velocities were insufficient to transport the sediment load supplied from upstream. This explained its tendency for siltation and requirement for frequent maintenance. On the basis of the geomorphological assessment, initial recommendations were proposed for morphological restoration of the channel to support the enhanced river environment envisaged in the catchment plan. It was further proposed that the viability of these initial recommendations should be examined further to determine their feasibility for a restoration project.

Table 2 Regime and Engineered Morphology of the River Blackwater, UK

Channel Parameter (1)	Regime Channel (2)	Engineered Channel (3)
Width (m)	6.3	12
Depth (m)	0.58	1
X-section area (m^2)	3.7	12
Mean velocity (ms^{-1})	1.0	0.3

In this application, the effective discharge was found to be the key to deriving values for the dimensions of a natural, regime channel appropriate to the current flow regime. This information was useful in highlighting the problems with the engineered channel and it indicated the starting point for detailed design of a restored channel.

Application 2: Channel Stability Assessment Using the SAM Hydraulic Package

Abiaca Creek, Mississippi has been monitored for five years as part of the DEC Monitoring Project by the U.S. Army Corps of Engineers Vicksburg District and Waterways Experiment Station. Based on observations of the thalweg profiles and channel morphology for this period, Watson *et al*. (1996) concluded that a study site (Site No. 6) on Abiaca Creek was in dynamic equilibrium. Data from this site were used by Watson *et al*. (1997) to develop a stable channel design procedure for sand-bed streams based on the minimum-slope extremal hypothesis (Chang, 1979).

The design procedure was developed and tested using hydrological and effective discharge data generated using the SAM hydraulic design package (Thomas *et al*., 1994) and HEC-6T (Thomas, 1996). SAM was applied to generate a series of 21 combinations of width, depth, and slope that satisfy water and sediment continuity for a given flow and sediment concentration. The morphology corresponding to the minimum slope was selected from the 21 combinations as the preferred stable channel design. The flow used in these computations was the effective discharge, calculated using the procedure presented here, with logarithmic class intervals.

15-minute discharges for a complete hydrological year were simulated for the preferred design and 9 of the other 20 alternative morphologies. For each condition, HEC-6T, a 1-D sediment routing model, was used to simulate expected changes to the initial channel geometry. Of the ten morphologies tested, the preferred design (based on the minimum-slope condition), resulted in the smallest change in channel slope (0.24%) and generated no change in bed elevation. On this basis, it was selected as the most stable design.

In this application, the effective discharge was found to provide a reasonable representation of the range of flows actually experienced by the stream, for the purpose of stable channel design.

Application 3: River Restoration Design

Determination of effective discharge is an initial step in the Waterways Experiment Station (WES) best practice hydraulic design procedure for restoring channels. The preferred cross-sectional geometry is a compound configuration composed of a primary channel, designed to carry the effective or 'channel forming' discharge, and an overbank area designed to carry the additional flow for a specific flood discharge. The effective discharge is calculated during a geomorphic assessment that is undertaken at a stable location upstream of the project reach. This ensures that the restored channel design will transport the sediment load from upstream with minimal net aggradation or degradation in the medium term. The procedure then equates the target bankfull discharge for the primary channel in the restored reach to the effective discharge.

Hydraulic geometry relationships appropriate to the type of channel are applied to determine a range of possible stable bankfull widths as a function of effective discharge within user-defined confidence limits (Soar *et al*., 1998). A selected width, within the confidence band, is then used, together with the

sediment load transported by the effective discharge, to determine a stable water surface slope and mean cross-sectional depth. This computation is accomplished using the SAM hydraulic design package (Thomas *et al*., 1994), through simultaneous solution of Brownlie's equations for bed material load transport and flow resistance. Design dimensions are reach-average values for a trapezoidal channel shape and they may then be modified to account for local variability in cross-sectional form between, for example, crossings and bendways in a meandering stream.

In this application, the effective discharge is required to enter the SAM package for stable channel design. Hence, an accurate and reliable effective discharge calculation is an essential precursor to application of the SAM package in channel restoration design.

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APPENDIX II. GLOSSARY

1.58-Year Return Period Discharge **($Q_{1.5}$)** - The *discharge* with a return period of 1.58 years, derived from the observed annual maximum flow series. Results of research on the use of $Q_{1.5}$ in dominant discharge analysis are reported by Hey (1975) for the UK and Leopold *et al*. (1964) for the USA.

2-Year Return Period Discharge **(Q_2)** - The *discharge* with a return period of 2 years, derived from the observed annual maximum flow series. Results of research on the use of Q_2 in American rivers are reported by Biedenharn *et al*. (1987).

Bankfull Discharge **(Q_b)** - The maximum *discharge* which can be contained within the channel without over-topping the banks. Leopold *et al*. (1964) proposed that it is this flow which is responsible for forming and maintaining the morphology of the channel. Bankfull stage refers to the water surface elevation during bankfull flow and can be identified from various criteria (Williams, 1978). Research papers reporting the use of Q_b include: Leopold and Wolman (1957), and Andrews (1979) in the USA; and Charlton *et al*. (1978), and Hey and Thorne (1986) in the UK.

Bed Load - A component of the *total sediment load* made up of sediment particles moving in frequent, successive contact with the bed (Bagnold, 1966). Transport occurs at or near the bed, with the submerged weight of particles supported by the bed. Bed load movement takes place by gravitational processes of rolling, sliding or saltation.

Bed Material Load - A portion of the *total sediment load* composed of grain sizes found in appreciable quantities in the stream bed. In gravel-bed rivers the bed material load moves as *bed load*, but in sand-bed streams significant quantities of bed material load move as *suspended load*.

Channel Forming Discharge - The *discharge* that most efficiently drives the fluvial processes responsible for forming and maintaining the main morphological features and dimensions of the channel. Synonymous with *dominant discharge*.

Design Discharge - The steady *discharge* used in the engineering design of a stable channel or flood defence scheme to define the upper boundary of the operating range of discharges for the project.

Discharge - The volume of water passing through a cross-section in a stream per unit time. It is usually expressed in cubic metres or cubic feet per second.

***Dominant Discharge* (Q_{dom})** - The single, steady *discharge* which would produce the same cross-sectional morphology, alluvial features, planform geometry, and dimensions as those generated by the actual flow regime (Inglis, 1949).

***Effective Discharge* (Q_e)** - *Discharge* class responsible for transporting the largest fraction of the bed material load in a stable channel over a period of years (Andrews, 1980, pp. 311). Defined by the peak in a histogram of bed material load (tons) versus discharge (cumecs) developed using the principles of magnitude and frequency analysis (Wolman and Miller, 1960).

Ephemeral Stream - Watercourse in which channel processes and morphology are significantly affected by the fact that the discharge of water is intermittent. To be comparable with the definition of a *perennial stream*, this may be taken as a water course which exhibits a measurable surface discharge less than 80% of the time (Osterkamp and Hedman, 1982).

Fine Material Load - A portion of the *total sediment load* composed of particles finer than those found in the stream bed, and usually the fraction finer than 0.062 mm. Often synonymous with *wash load*.

Flow Doing Most Work - A steady *discharge* which performs the most geomorphic work, where work is defined in terms of sediment transport (Wolman and Miller, 1960).

Flow Duration Curve - A graphical representation of the percent of time (x-axis) that a specific discharge (y-axis) is equalled or exceeded during the period of record for which the curve was developed.

Hydraulic Geometry - A geomorphic expression introduced by Leopold and Maddock (1953) to describe the morphology of an alluvial river as a function of dominant discharge. The concept is similar to *regime theory*, but differs in the way that the *dominant discharge* is expressed. With respect to the hydraulic geometry of an alluvial river, the *dominant discharge* is the single flow event which is representative of the natural sequence of events which actually occur. Regime theory was developed for canals, which do not experience a range of flows. Hence, the *dominant discharge* for regime theory is the steady, operating discharge.

***Mean Annual Discharge* (Q_{ma})** - Yearly-averaged *discharge*. Papers reporting research involving Q_{ma} include: Carlston (1965, 1969); Dury (1964); Leopold and Maddock (1953); Schumm (1971).

***Mean Annual Flood* ($Q_{2.33}$)** - *Discharge* corresponding to the probability of exceedance of the mean annual flood event in a Gumbel extreme value type 1 probability distribution (EV1) derived from the observed annual maximum flow series. This event has a recurrence interval of 2.33 years. Papers reporting research involving $Q_{2.33}$ include Brush (1961) and Ferguson (1973).

Measured Load - A portion of the *total sediment load* measured by conventional suspended load samplers. Includes a large proportion of the *suspended load* but excludes that portion of the suspended load moving very near the bed (that is, below the sampler nozzle) and all of the *bed load*.

***Most Probable Annual Flood* ($Q_{1.58}$)** - The *discharge* corresponding to the probability of exceedance of the modal annual flood event in a Gumbel extreme value type 1 (EV1) derived from the observed annual maximum flow series. This event has a recurrence interval of 1.58 years. Papers reporting research involving $Q_{1.58}$ include Woodyer (1968) and Dury (1973).

Perennial Stream - A stream which exhibits a measurable surface discharge more than 80%of the time (Osterkamp and Hedman, 1982).

Regime Theory - A self-formed alluvial channel is in regime if there are no net changes in discharge capacity or morphology over a period of years. The concept was originally developed by engineers designing canals to convey a steady discharge with neither erosion or siltation in India and Pakistan (Kennedy, 1895; Lindley, 1919) and, later in North America (Blench, 1957).

Sediment Concentration - Concentration of sediment in the stream represented by the ratio of *sediment discharge* to the water *discharge*. Usually expressed in terms of milligrams per litre or parts per million (ppm). It is normally assumed that the density of the water-sediment mixture is approximately equivalent to the density of the water.

Sediment Discharge - Mass of sediment that passes through a cross-section in a stream per unit time. Usually expressed in kilograms per second or tons per day.

Sediment Rating Curve - Graphical representation of the non-linear relationship between *discharge* (x-axis) and *sediment discharge* (y-axis).

Suspended Bed Material Load - A portion of the bed material load that is transported in suspension within the water column.

Suspended Load - A component of the *total sediment load* made up of sediment particles moving in continuous suspension within the water column. Transport occurs above the bed, with the submerged weight of particles supported by anisotropic turbulence within the body of the flowing water.

Total Sediment Load - Total mass of granular sediment transported by the stream.

Unmeasured Load - A portion of the *total sediment load* that passes beneath the nozzle of a conventional suspended load sampler, by near-bed suspension and as *bed load*.

Wash Load - A portion of the *total sediment load* composed of grain sizes finer than those found in appreciable quantities in the stream bed. In sand-bed streams wash load moves as *suspended load*, but in boulder-bed rivers the wash load may include gravel which moves as *bed load*. The sum of *bed material load* and *wash load* makes up the total sediment load.